Feasibility Report Appendixes

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American River Watershed Investigation, California

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American River Watershed Investigation, California

FEASIBILITY REPORT

LIST OF APPENDIXES

American River Watershed Investigation, California

APPENDIX N

Designs and Cost Estimates

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

BASIS OF DESIGN AND COST ESTIMATES

GENERAL

The following chapters present the designs and cost estimates used for the American River Watershed Investigation feasibility report. Because the components of the proposed alternatives and the final selected plan are very different, this appendix is divided into chapters which address the different components. Chapter 1 presents the analysis used for the levee components, Chapter 2 presents the design for a pump station on the Natomas East Main Drainage Canal (done by A/E contract), Chapter 3 presents the analysis for the dam portion, and Chapter 4 is the cost estimate for the final selected plan. Each chapter first describes the alternatives used in determining the National Economic Development (NED) plan. Costs developed for these alternatives are presented in the individual chapters. These initial alternative designs and costs are not M-CACES level designs or costs. They are designs and costs adequate to give enough information to be used for identification of the NED plan. After the NED plan was identified, this information along with other information was used to determine the selected plan. The selected plan was then analyzed using an M-CACES approach. This involved additional detailed design and a more thorough approach to the cost estimate using an M-CACES analysis. This additional analysis for the selected plan along with guidance from the Feasibility Resolution Conference, the Technical Resolution Conference, and the Project Guidance Memorandum resulted in design differences between the alternatives analyzed and the selected plan. The selected plan is described at the end of each chapter. Differences between the alternatives analyzed and the selected plan are described there. Chapter 4 gives the M-CACES cost estimate for the selected plan.

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

DOCUMENTATION REPORT

APPENDIX N

DESIGNS AND COST ESTIMATES

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

APPENDIX N

CHAPTER 1

BASIS OF DESIGN AND COST ESTIMATES

LEVEE ALTERNATIVES

OCTOBER 1991

BASIS OF DESIGN AND COST ESTIMATES
LEVEE ALTERNATIVES

TABLE OF CONTENTS

LIST OF TABLES

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

CHAPTER 1

BASIS OF DESIGN AND COST ESTIMATES
LEVEE ALTERNATIVES

GENERAL

This report will describe the information used in determining alignment, freeboard, quantities, and costs for the levee components of the alternatives considered and the levee component of the selected plan. Levee work is proposed in the Sacramento area downstream of Folsom Dam. Modifications investigated consist of raising existing levees and constructing some reaches of new levee. Levee alternatives for different levels of flood protection and levees proposed to protect only portions of some flood prone areas were investigated. Plate 1 shows the levees investigated.

ALIGNMENT

Most levee work consists of raising existing levees with the alignment being determined by the existing alignment. Plate 1 shows the levees which currently exist in the Sacramento area and the alignment of the few proposed new levees. New levee alignments were sited to provide protection to flood prone areas and in an attempt to minimize relocations and disruption to existing development. These levees were terminated at points with elevations adequate to prevent flanking. The new levees investigated include cross levees in the Natomas area, a levee north of Dry Creek from Natomas East Main Drain (NEMD) to Rio Linda Boulevard, an extension of the Dry Creek south levee to Rio Linda Boulevard, and a levee along the south side of the American River upstream of Mayhew Drain.

MAPPING AND TOPOGRAPHY

Existing levee topography was determined from USGS Quadrangle sheets, recently surveyed levee profiles, asbuilt construction plans, and cross sections surveyed for hydraulic studies in the area. In those reaches where cross sections were not available for existing levees, top widths were field verified and side slopes were assumed to be the same as shown on original construction plans, in most cases 1V to 2H landside and 1V to 3H waterside. Where regularly spaced cross sections were not available, in many instances a few schematic sections had been developed during the levee stability analysis accomplished for the Sacramento River Levee Evaluation under Inspection of Completed Works. Field observation determined that most of the levees are consistent in shape. Therefore, the few sections that are provided serve to adequately represent long reaches of levee. Table N-1-1 presents information on

TABLE N-1-1

EXISTING LEVEE INFORMATION AND SOURCES

REACH	TOP WIDTH (FT)	LEVEE PROFILE	SIDESLOPES LAND SIDE	SIDESLOPES WATER SIDE	HEIGHT (FT)
NEMD EAST LEVEE					
AM R TO ARCADE CR	20	1/	2:1	3:1	9-19
ARCADE CR TO DRY CR	15	1/	2:1	3:1	7-16
NEMD WEST LEVEE					
EL CAMINO RD TO MAIN ST	26-32	2/	2:1	3:1	10-19
MAIN ST TO NEMD PLUG	25	3/	2:1	3:1	5-15
ARCADE CREEK					
SOUTH LEVEE	12-20	4/	2:1	3:1	5-20
NORTH LEVEE	12-20	4/	2:1	3:1	5-13
DRY CREEK					
SOUTH LEVEE	20	5/	2:1	3:1	4-10
PLEASANT GROVE CREEK CANAL	25	6/	2:1	3:1	10-15
NATOMAS CROSS CANAL (NCC)					
SOUTH LEVEE	22-35	7/	2:1	3:1	12-33
NORTH LEVEE	25-30	8/	2:1	3:1	14-28
SACRAMENTO RIVER EAST LEVEE					
FREMONT WEIR TO ELVERTA RD	25-40	8/	2:1	3:1	7-19
ELVERTA RD TO DEL PASO RD	25-40	8/	2:1	3:1	5-19
DEL PASO RD TO AMERICAN R	25-60	8/	2:1	3:1	5-17
AMERICAN R TO FREEPORT	20	8/	2:1	3:1	10-24
SACRAMENTO RIVER WEST LEVEE					
FREMONT WEIR TO ELVERTA RD	25-50	8/	2:1	3:1	9-18
ELVERTA RD TO DEL PASO RD	20-40	8/	2:1	3:1	11-20
DEL PASO RD TO AMERICAN R	20-40	8/	2:1	3:1	11-16
AMERICAN R TO FREEPORT	20-30	8/	2:1	3:1	10-27
SACRAMENTO BYPASS					
NORTH LEVEE	25-30	8/	2:1	3:1	10-27
SOUTH LEVEE	20-30	8/	2:1	3:1	11-26

TABLE N-1-1 (Cont.)

YOLO BYPASS EAST LEVEE					
FREMONT WEIR TO SAC BYPASS	20	8/	2:1	3:1	14-21
SAC BYPASS TO LISBON	20	8/	2:1	3:1	12-24
YOLO BYPASS WEST LEVEE					
FREMONT WEIR TO I-5	20	8/	2:1	3:1	13-18
I-5 TO PUTAH CREEK	20	8/	2:1	3:1	7-22
AMERICAN RIVER					
NORTH LEVEE	20-25	9/	2:1	3:1	7-21
SOUTH LEVEE	20-25	9/	2:1	3:1	3-16

FOOTNOTES FOR TABLE N-1-1

1/ From channel cross sections surveyed in 1988 used in 1988 City of Sacramento Flood Insurance Study.

2/ From levee profile surveyed in May 1988 for Reclamation District 1000

3/ From channel cross sections surveyed in 1988 used in 1988 City of Sacramento Flood Insurance Study.

4/ From channel cross sections surveyed in 1987 used in 1988 City of Sacramento Flood Insurance Study.

5/ From Ortho Photo Contour Maps developed in 1986 (1"=200', 2' Contours) for Dry Creek Feasibility Study.

6/ From channel cross sections surveyed in 1988 in 1988 City of Sacramento Flood Insurance Study.

7/ From levee profile surveyed in Nov 1987 for Reclamation District 1000.

8/ From levee profile surveyed in Jun 1988 by the California Department of Water Resources.

9/ From channel cross sections surveyed in October 1987 used in 1988 City of Sacramento Flood Insurance Study.

existing levee topography and sources of information. Ground surface elevations and information for the new Natomas cross levees were taken from USGS quadrangle sheets. Ortho Photo Contour Maps developed for the Dry Creek Feasibility Study were utilized for the new levees along Dry Creek. Cross sections surveyed in October 1987 were used for the new levee of the American River.

During PED a mapping program will be done to develop topography along the levees to be modified and along the alignment of new levees. Maps will be developed on a scale of 1"=50' and will show all physical features within 100 feet of both sides of the levee centerline. A terrain model will be developed for use with the Intergraph System.

DESIGN DETAILS

Design levee sections were chosen to remain the same as used in past design for the existing levees. These sections have performed adequately and the levee reevaluation studies demonstrated that these sections would have adequate stability if raised only a few feet. Analysis of levee stability after the proposed modifications is given in Appendix M, Chapters 1, 2, and 3. In many cases the existing levees were constructed with top widths wider than the design widths. Results are that in raising the existing levees using the design widths, work will only have to be done on top of many of the existing levees. In only a few cases will work extend past the existing toe of the levees. There are reaches of levee which have public roads on top. Where there are public roads and the levees must be modified, top widths will be to the minimum safe roadway widths for these type of roads (minimum 28 feet) or to current roadway widths if they exceed minimum roadway widths. New levee sections have top widths chosen the same as existing levees if being extended or top widths adequate for the new levee height and design flows being contained. Levees without public roads will have a 14 foot wide patrol road on top if the design width permits. Some levees have only a 12 foot crown width and these will have patrol roads across the entire crown.

In determining whether new levee fill would be on the landside or waterside, consideration was given to how much fill was being done and impacts to utilities, relocations, and development. Excessive fill in the waterway could significantly reduce conveyance with a resultant rise in design water profiles. Along the NEMD East Levee a railroad line would be impacted with a landside fill. In order to avoid relocating a significant stretch of railway, it was decided to fill on the waterside in this reach. Levee raising is small (maximum one foot) and encroachment into the waterway is insignificant. Table N-1-2 lists the levee design details for the different levee reaches.

FREEBOARD

Design freeboard for the different levees was developed to accomplish the design objectives and to allow for any uncertainties in water surface profiles. The freeboard was determined using EM 1110-2-1601 and ETL 1110-2-299 as guidance.

TABLE N-1-2

DESIGN DETAILS FOR LEVEE REACHES

REACH	TOP WIDTH (FT)	SIDESLOPES LAND SIDE	SIDESLOPES WATER SIDE	SIDE OF FILL
NEMD EAST LEVEE				
AM R TO ARCADE CR	20	2:1	3:1	WATERSIDE
ARCADE CR TO DRY CR	15	2:1	3:1	WATERSIDE
NEMD WEST LEVEE				
EL CAMINO RD TO MAIN ST	20	2:1	3:1	LANDSIDE
MAIN ST TO NEMD PLUG	28	2:1	3:1	LANDSIDE
ELVERTA RD TO SANKEY RD	28	2:1	3:1	LANDSIDE
ARCADE CREEK				
SOUTH LEVEE	12	2:1	3:1	LANDSIDE
NORTH LEVEE	12	2:1	3:1	LANDSIDE
DRY CREEK				
SOUTH LEVEE	20	2:1	3:1	WATERSIDE
NORTH LEVEE	20	2:1	3:1	NEW LEVEE
PLEASANT GROVE CREEK CANAL	28	2:1	3:1	LANDSIDE
NATOMAS CROSS CANAL (NCC)				
SOUTH LEVEE	20	2.5:1	3:1	LANDSIDE
NORTH LEVEE	20	2:1	3:1	LANDSIDE
SACRAMENTO RIVER EAST LEVEE				
FREMONT WEIR TO ELVERTA RD	30	2:1	3:1	LANDSIDE
ELVERTA RD TO DEL PASO RD	30	2:1	3:1	LANDSIDE
DEL PASO RD TO AMERICAN R	30	2:1	3:1	LANDSIDE
AMERICAN R TO FREEPORT	20	2:1	3:1	LANDSIDE
SACRAMENTO RIVER WEST LEVEE				
FREMONT WEIR TO ELVERTA RD	20	2:1	3:1	LANDSIDE
ELVERTA RD TO DEL PASO RD	20	2:1	3:1	LANDSIDE
DEL PASO RD TO AMERICAN R	30	2:1	3:1	LANDSIDE
AMERICAN R TO FREEPORT	20	2:1	3:1	LANDSIDE
SACRAMENTO BYPASS				
NORTH LEVEE	30	3:1	4:1	LANDSIDE
SOUTH LEVEE	30	3:1	4:1	LANDSIDE

TABLE N-1-2 (Cont.)

YOLO BYPASS EAST LEVEE				
FREMONT WEIR TO SAC BYPASS	20	3:1	4:1	LANDSIDE
SAC BYPASS TO LISBON	20	3:1	4:1	LANDSIDE
YOLO BYPASS WEST LEVEE				
FREMONT WEIR TO I-5	20	3:1	4:1	LANDSIDE
I-5 TO PUTAH CREEK	20	3:1	4:1	LANDSIDE
AMERICAN RIVER				
NORTH LEVEE	20	3:1	4:1	WATERSIDE
SOUTH LEVEE	20	3:1	4:1	WATERSIDE

Natomas Area

This discussion presents the freeboard determination for the NEMD, Arcade Creek, Dry Creek, Pleasant Grove Creek, NCC, and the Sacramento River. Freeboard for this area was established to provide adequate protection for the type of areas being protected based on the reliability of the profiles provided. Original freeboard for the Natomas area levees was 3 feet and 2 1/2 feet. At the time of the existing levee design, this area was not as developed as currently. The minimum freeboard determined to be adequate for the urban area under consideration is 3 feet. This amount could be increased to account for uncertainties or to provide levee superiority in this area. Most reaches of these levees protect against backwater from the American and the Sacramento Rivers. As such, the flows in these reaches generally have low velocities. These low velocities reduce the uncertainties associated with potential blockage of bridge openings and sudden hydraulic expansions and contractions. The backwater reaches are the NEMD, Arcade Creek, Dry Creek, NCC, and the Pleasant Grove Creek Canal.

Design water surface profiles were developed using the DWOPER hydrologic model and the HEC-2 hydraulic model which were developed using recently surveyed cross sections and calibrated for the 1986 flood which was the flood of record in most reaches. Because of this, the design profiles are considered to be very reliable for the design flows being considered. No additional freeboard above the minimum freeboard is considered necessary to account for uncertainties in design profile calculations. In the Sacramento and American River reaches where velocities could be appreciable, bridge crossings are few and bridge clearance and openings are large enough to prevent drift blockage. Inspection of velocities in these reaches did not find any areas with significantly high velocities. Again no additional freeboard is considered necessary for these bridge backwater uncertainties. The elevations for the Sacramento and American Rivers in this area are largely influenced by the Fremont Weir and Sacramento Weir. Sacramento River elevations in the Natomas Area are particularly influenced by flow over both weirs. The effects of the weirs are such that large changes

in discharge do not cause large changes in elevation and these effects produce flat rating curves for the Sacramento River. Elevations at the mouth of the American River are largely influenced by the Sacramento Weir and floodway conveyance. The American River floodway is wide at its lower end and this in concert with weir flow effects also produce flat rating curves in this area for the American River. These flat rating curves indicate that no increase in minimum freeboard is necessary to adjust for uncertainties in design flows.

In addition to low velocities in the reaches, there are no sharp curves. Therefore there is no need to increase freeboard to account for superelevation of flows. There is no history of wave wash in these areas and fetch lengths are very short so that wave set or runup is not significant enough to warrant increases in design freeboard.

Levee superiority was given consideration for the Natomas area. The purpose of superiority is to design freeboard in such a manner that should design flows be exceeded, the levee system will fail in a way to cause the least catastrophic effects. The Sacramento Area levee system is complex and several areas such as the Natomas area are surrounded by levees such that levee failure at any point will cause filling of these enclosed areas in a bathtub effect. Natomas could be flooded from either the American or Sacramento Rivers. Allowing failure to occur first from either of these two sources will not reduce chances of failure from the other. If failure is allowed to occur along any reach of the Natomas Levees, the entire area will still flood due to the bathtub affect mentioned above. Leveed areas north of the NCC and west of the Sacramento River currently have less development than the Natomas Area. However, even though failure of these areas would reduce pressure on the Natomas Area, these leveed areas are under different political jurisdictions, Levee Maintenance Districts, and Counties, and these entities would not allow their areas to be sacrificed for the benefit of the Natomas residents. They have already expressed concern on how Natomas levee modifications may affect the levees protecting their areas. No superiority reaches could realistically be identified and minimum freeboard for the Natomas area was not altered in an attempt to provide superiority. Changes in water surface elevations are not sudden in this area and should it become apparent that design flows are going to be exceeded, there would be some time to warn the protected areas.

Based on the discussion above final design freeboard for the Natomas area was set at three feet for all levee reaches.

Lower American River

One measure used in several alternatives is to raise and strengthen the existing Lower American River levees to withstand objective releases from Folsom Dam of 130,000 cfs and 180,000 cfs. The current objective release is 115,000 cfs. These levees convey flows through the highly urbanized area of Sacramento. The last units of the American River levees were designed with freeboard to provide 5-feet over 115,000 cfs or 3-feet over 152,000 cfs. The height of the existing levees was determined by the higher of these two criteria.

The profiles developed for the Lower American River were based on 1987 surveyed cross sections which covered the flood plain from levee to levee. These profiles were calibrated using high water marks for the 1986 flood event. The 1986 peak flow along the Lower American River was approximately 130,000 cfs. With the calibration of the hydraulic model to these 1986 high water marks, the 130,000 cfs profile is considered very reliable. Differences in stage between the 130,000 and 180,000 cfs profiles vary from three to five feet. Because of the recently surveyed cross sections used, the 180,000 cfs profile is also considered to be reliable, but not to the degree as the 130,000 cfs profile. The 180,000 cfs profile is considered to be less reliable than the 130,000 cfs profile because it much more exceeds the discharges used for calibration. A minimum of three foot of freeboard was selected for the urban area being protected. No extra freeboard is considered necessary to correct for uncertainties in profile calculations for the 130,000 cfs profile because it very nearly equals the 1986 discharges used for calibration. An additional foot of freeboard for the 180,000 cfs profile is being added due to less reliability for the computations of this higher discharge and the fact that higher stages are being contained between levees in an highly urbanized area.

The main objective of levee freeboard is to convey the design flows with a high degree of safety through the area of protection. Another objective, which can impact the design freeboard selected, is levee superiority. This is an attempt to design the levee is such a manner that flows exceeding the design flow will fail the levee in an area or in a manner that will cause the least amount of damage and have the least likelihood of causing loss of life.

The 3-feet and 4-feet freeboards mentioned above would be adequate for safety. However, to insure failure in the least damaging location, as described in ETL 1110-2-299 and ER 1110-2-1405, modifications to these values may be necessary. It is difficult to pick an area of least damage in the Lower American River levee system. Development occurs right up to the levees in all areas. After investigating flow patterns, it appears that allowing the levee to overtop first above Mayhew Drain on the south side would be less catastrophic. If flows exit the levee system on the south side, they flow away from the river and do not return to the American River levee system. These flows will continue through downtown Sacramento and other residential areas and spread over a large area south of Sacramento. Levees in this reach will be less tall than the levees in the lower end of the system. These flows will cause damage and flood large areas, but there should be no catastrophic wall of water with which to contend. If the flows overtop the levees along the north levee they will be contained in a flow path bounded by levees on the left and higher ground on the right. The flows will continue west until they begin to pond behind high ground in the area of Cal Expo and the east levee of the Natomas East Main Drain. These ponded flows would probably break back into the levee system to continue downstream and threaten to exceed design capacity of the lower end of the levee system. By designing the system to first fail along the southern levee, flows which exceed the design flow will be allowed to flow out of the system and not threaten the integrity of the downstream

system.

The minimum existing levee height above the 130,000 cfs profile in the downstream reaches is 5-feet. All other reaches of the project levees exceed the 130,000 cfs profile by six or more feet. The only areas that will need to be raised for the 130,000 cfs flow is the reach of new levee on the south bank upstream of Mayhew Drain. In this reach the freeboard for the 130,000 cfs profile will be 4-feet to ensure levee superiority of the downstream levee system.

The minimum existing levee height above the 180,000 cfs profile is one-foot in the Cal Expo reach. The north levee will have to be raised from here upstream to safely convey the 180,000 cfs flow. As stated above, 4-feet of freeboard is considered adequate for this flow. Again the reach of new levee on the south side above Mayhew Drain will be designed with 4 feet of freeboard. Therefore, the freeboard for the 180,000 cfs design will be increased to five feet in all other reaches to insure superiority.

Consideration was given to increasing levee freeboard at bridges. Most of the bridges crossing the American River span the flood plain from levee to levee and distances between bridge piers appear to be wide enough to prevent much debris from accumulating. Some of the railroad trestles are older wooden trestles which are closely spaced. Additional freeboard of 1-2 feet over the minimum freeboard already exist in these areas. Therefore no extra freeboard is considered necessary at potential troublesome bridge crossings.

Because flood flows are regulated by Folsom Dam, the rate of change of flows would not be sudden. Flows would tend to increase at a prescribed rate to the objective release and remain there until flows into Folsom Lake decreased. Flows would not increase above the design flows until flood storage space in Folsom was exhausted. There would be some warning time before this occurred to prepare for flows which might exceed the design flows.

Based on the above, the freeboard used for the 130,000 and 180,000 cfs American River levee designs are 4-feet for the south levee reach above Mayhew Drain and 5-feet for all other reaches.

Sacramento and Yolo Bypass

Design water surface profiles for the bypasses were developed using the DWOPER hydrologic model as described in Appendix K. These profiles were calibrated for the 1986 flood of record and are considered to be very reliable for the design flows being considered. No additional freeboard above the minimum freeboard is considered necessary to account for uncertainties in design profile calculations. Rating curves for the Yolo and Sacramento Bypass are very flat in the high discharge range. This is due to the large conveyance they provide and because of upstream levee failures which store much of the upper frequency storms. Therefore, no increase above minimum freeboard is considered necessary to account for uncertainties in flood flow determinations.

The minimum freeboard is first established at 3 feet. An additional three feet over normal freeboard for the bypasses is provided for wave runup. Because of the width of the Yolo Bypass, substantial waves can

be generated by winds during floods. The additional freeboard will prevent these waves from overtopping the levees and causing a wave erosion failure. Any modifications to the Sacramento or Yolo Bypass levees will be designed with 6 feet of freeboard.

Table N-1-3 lists the design freeboard used for the levee reaches being considered.

TABLE N-1-3

DESIGN FREEBOARD FOR LEVEE REACHES

REACH	DESIGN FREEBOARD (FEET)
NEMD EAST LEVEE	3
NEMD WEST LEVEE	3
ARCADE CREEK	3
DRY CREEK	3
PLEASANT GROVE CREEK CANAL	3
NATOMAS CROSS CANAL	3
SACRAMENTO RIVER EAST LEVEE	3
AMERICAN RIVER	
South Levee above Mayhew Drain	4
All Other Levee Reaches	5
YOLO BYPASS	6
SACRAMENTO BYPASS	6

SEDIMENTATION AND EROSION PROTECTION

No sedimentation analysis has been done for the floodways under consideration. The existing projects have operated for over 30 years with no indication of sedimentation problems which would reduce conveyance. As was mentioned in the discussion on freeboard design, design elevations along the NEMD, Arcade Creek, Dry Creek, and NCC are influenced mostly by backwater effects from the American and Sacramento Rivers. These backwater elevations would be minimally affected by any sedimentation in these reaches even if it were to occur. The American River shows no evidence of sedimentation problems. Degradation along this reach of river would be expected due to the effects of Folsom Reservoir upstream. The Sacramento River has also not shown any sedimentation impacts. Sedimentation is not considered to be a problem for the levee system under design.

No bank or levee protection is required along the reaches of NEMD, Arcade Creek, Dry Creek, and the NCC. The existing channel banks and levee slopes support a good growth of vegetation, have been in existence for 30 years with negligible erosion, and channel velocities are low. Any levee raising which is done on the waterside will be seeded to

prevent erosion. The American River alternatives for 130,000 and 180,000 cfs objective releases will require extensive erosion protection which is detailed in Appendix M, Chapter 4. Erosion along the Sacramento River is under constant surveillance and problem areas identified are corrected under the authority of the ongoing Sacramento River Bank Protection Project. No sites in the study area have been identified for bank protection. Erosion activity will not be increased by the proposed flood control project.

INTERIOR DRAINAGE

Natomas Area

An extensive interior drainage system already exists for the Natomas levee system. The current interior drainage system has operated adequately in the past. Raising of the levees around the Natomas area and along the existing American River levees would not change the current operation of the existing systems and no modifications are recommended for the levee alternatives being considered.

American River

New levees built on the south bank of the American River upstream of Mayhew Drain cause drainage from Mayhew Drain to become a consideration. These new levees are a part of the alternatives identified as 100-year, 130,000 cfs Folsom objective release and 150-year alternatives. These alternatives are described in more detail later in this chapter. The American River alternatives include a pumping plant at Mayhew Drain to handle flows during high stages of the American River. This plant was considered to operate peak on peak due to the lack of area for sump storage in the Mayhew Drain area. Required pumping capacity was conservatively estimated at 2000 cfs.

Natomas East Main Drain

NEMDC Pump Station. -

Design - Levee modifications in the Natomas area include a levee "plug" in the NEMD just upstream of where Dry Creek enters the NEMD. This levee crosses the NEMD and continues north of Dry Creek until high ground can be reached. Backwater is prevented from entering the upper reaches of the NEMD by this levee and thus precludes the need for levee raising in the upper reaches. In order to prevent with project conditions from being worst than without project conditions, the flows and storage behind this "plug" levee must be addressed. Runoff hydrographs for the drainage area behind this levee include flows which spill down from the Pleasant Grove Creek drainage area and flows diverted from the Natomas area by raising Sankey Road as described in the alternatives. In addition, there is a Natomas area pump which pumps interior runoff into the NEMDC. A ponding analysis of the area behind this plug levee determined that a pumping plant with a 700 cfs capacity

is required to prevent interior elevations from exceeding without project conditions and to keep elevations low enough to eliminate the need for levee work on these upstream levees. Additional design criteria is the pump must be able to pump against a maximum head of 10 feet and should be capable of pumping at an interior flood elevation of 28. The pumping plant was designed by A-E contract and is described in Chapter 2 of this appendix. As a part of the pumping plant, it is desirable to have a means of passing low flows to keep actual pumping operation to a minimum. A method for accomplishing this is to include low flow sluices as a part of the pumping plant.

Low Flow Sluices - The first trial objective was to size the sluices to pass the 100-year flow with insignificant backwater effects. This proved to be not feasible because the normal backwater elevation of the 100-year flow is 29.7. This elevation would cause pumping to begin at the station. The pumping station is designed to begin pumping at elevation 28 and to have all pumps operating at elevation 30. The second and adopted objective was to provide low flow sluices which would pass the majority of summer low flows and winter flows and to determine an operating scheme which would prevent backwater from the Sacramento and American Rivers from encroaching into sump storage needed upstream of the pumping plant. The low flow sluices provided are two 8' wide by 4' high concrete box culverts with an invert elevation of 24. These culverts will have two positive closure sluice gates each. This is to provide an emergency gate in case the primary gate is unable to close. Without a guarantee that the pumping plant can prevent backwater from moving upstream, this measure would not be effective in flood control protection.

Operation - The operating scheme is to leave the low flow sluices open until the Sacramento River I Street Gage reaches Warning Stage, Elev. 25.0, or the pumps at the pump station begin to operate, Elev 28 at the pumping plant. This criteria will allow the sluices to pass approximately 300 cfs before being closed. This should provide for most discharges. Once closed, the sluices will remain closed until the I Street Gage drops to elevation 23 and the tailwater elevation at the pumping plant is less than elevation 25. The sluices will then be opened to allow low flows to pass. Interior elevations will be automatically monitored and the pumps activated and the sluices closed automatically once elevations reach the above described elevations. In addition the system will automatically notify a central agency that the station has gone into operation. This agency should send a person to the station to be sure that it is in fact operating correctly. As the pumps begin operating they will be sequenced such that the pumping capacity will be gradually increased until maximum capacity is reached at elevation 30.

Natomas Cross Levees

The proposed cross levee alternatives through the Natomas Area will impact the existing interior drainage system. These levees will prevent

drainage flows and irrigation flows from continuing to irrigation and pumping points. Where these levees intercept drainage flows, provisions were included to drain these flows to other existing pumps and increase the capacity of those existing pump stations if required. Irrigation flows would pass through the levee or be pumped over the levee.

ALTERNATIVE PLANS; DESCRIPTION AND COST

The levee alternatives investigated will provide different levels of protection to the Sacramento Area. One sub-alternative, the Natomas Cross Levees, investigated providing protection to only portions of a flood prone area, in this case the Natomas Area. This sub-alternative could be used with any of the other levee alternatives. The alternatives investigated include 100-year, 150-year, 200-year, and 400-year levels of protection. The levee plans would be in combination with different increases in upstream storage and different new objective releases from Folsom. Table N-1-4 shows the assumptions for each alternative. Plate 1 shows the locations of the levees investigated.

TABLE N-1-4

ASSUMPTIONS FOR DIFFERENT LEVEE ALTERNATIVES

ALTERNATIVE	UPSTREAM STORAGE	FOLSOM OBJECTIVE RELEASE
100-YEAR	INCREASE FOLSOM FLOOD CONTROL STORAGE BY 190,000 AC-FT	115,000 CFS
100-YEAR	NO INCREASE	130,000 CFS
150-YEAR	INCREASE FOLSOM FLOOD CONTROL STORAGE BY 250,000 AC-FT	180,000 CFS
200-YEAR	ADDITIONAL 545,000 AC-FT FLOOD CONTROL STORAGE UPSTREAM OF FOLSOM	115,000 CFS
400-YEAR	ADDITIONAL 890,000 AC-FT FLOOD CONTROL STORAGE UPSTREAM OF FOLSOM	115,000 CFS

Design water surface elevations were developed from DWOPER runs described in Appendix K and from HEC-2 runs. These HEC-2 runs were done using the hydraulic models developed for the recent City of Sacramento Flood Insurance Study performed by the Sacramento District. The design

water surface elevations and the design freeboards established the design grades for the different levee alternatives. Because water surface elevations in the Natomas area are mainly caused by backwater, there is little difference in design grades for alternatives which have the same objective release out of Folsom. Only when the objective releases are changed are there large differences in quantities and costs. Some of the alternatives contain the same flood control measures. These measures will be described in the first alternative in which they appear and this description referenced in subsequent alternative descriptions. The following information will discuss the different aspects of the levee alternatives.

Natomas Cross Levees

These sub-alternatives were designed to protect only a portion of the Natomas Area. These levees would cross the Natomas Area at one-third and two-third alignments. The levee alignments are shown on Plate 1. The cross levees would protect only those portions of Natomas which are currently developed. The one-third cross levee alignment would begin from the Sacramento River east levee at about River Mile 66.5 and continue east across Natomas just north of Del Paso Road. It would intercept the NEMD west levee at about River Mile 7.5. Along with this cross levee, the southern portions of the NEMD and Sacramento River levees would be raised, if necessary, to provide protection to the area within the newly ringed area. This cross levee is approximately 6.1 miles long. Right of way would be acquired to 5 feet from the toe of each side of the levee with real estate requirements being about 180 acres. The top of levee elevation is 42 feet NGVD, which would give a maximum levee height of 29 feet. Del Paso Road and Interstate Highway 5 would have to be ramped over the levee. Ramp heights would be approximately 20 feet. Two major drainage canals would be impacted as well as six irrigation ditches. The two-thirds cross levee alignment would begin from the Sacramento River east levee at about River Mile 75 and continue east across Natomas just north of Elverta Road. It would intercept the NEMD west levee. Again, the southern portions of the NEMD and Sacramento River levees would be raised, if necessary, to provide protection to the area within the newly ringed area. This cross levee is approximately 6.5 miles long. Right of way would be acquired to 5 feet from the toe of each side of the levee with real estate requirements being about 170 acres. The top of levee elevation is 44 feet NGVD, which would give a maximum levee height of 27 feet. Powerline Road and State Highway 99 would have to be ramped over the levee. Ramp heights would be approximately 20 feet for Powerline Road and 26 feet for Highway 99. One major drainage canal would be impacted and seven irrigation ditches. Table N-1-5 gives the construction cost for the cross levees of the Natomas Cross Levee alternatives.

100-Year, 115,000 CFS Objective Release

This alternative would involve raising the levees where necessary in conjunction with the stated objective release and raising opposite

levees to mitigate for hydraulic impacts to achieve 100-year level of protection. Most levee raising would be in the Natomas area. Hydraulic mitigation would be accomplished by assuring that no area's flooding frequency would be worse after the alternative levees were modified. This would be assured by raising opposite levees and backwater levees where necessary to restore other protected area's flood frequency to at least the same level as existed before levee modifications were done. Table N-1-6 describes the modification work required for this alternative. Plate 1 can be used to locate where the work will occur.

Portions of the east levee of the NEMD will be raised from Arcade Creek to Dry Creek. This work will be done on the waterside with the new toe of levee encroaching no more than 5 feet into the existing canal and on the average only 2 feet. Most of the work on the west levee NEMD will be done on the top of the levee. The Main Avenue Bridge will have to be replaced. This relocation will involve a high level separated grade crossing. This is necessary to prevent extensive alterations to the railroad which runs alongside the east levee of the NEMD.

Sankey Road is a low spot in the west levee of the NEMD. This reach of levee will be raised to prevent flood water from flowing into the Natomas area as occurred during the 1986 flood. Preventing this flow of water however, causes hydraulic impacts in the southern portion of the Pleasant Grove area. Water surface elevations would be increased by approximately 0.4 foot. To mitigate for this, a training channel will be excavated in the upper reach of the NEMD to convey the blocked flood flows to the new pumping plant at the NEMD plug. This channel will be trapezoidal in shape and grassed lined. The channel will pass under railroad maintenance tracks in this vicinity and box culverts will be provided under this track for the channel. Along with the channel a reach of the west levee of the NEMD will be raised downstream of Sankey Road. Raising this levee will require the public road on top of the levee to be replaced.

Only short reaches of the south and north levees of Arcade Creek will have to be modified. These reaches are just downstream of Marysville Boulevard. This is the point where flood flows in Arcade Creek begin to dominate over the backwater effects caused by the NEMD.

The upper end of the existing south levee of Dry Creek will have to be raised and an extension of this levee provided to high ground at Rio Linda Boulevard. A new levee north of Dry Creek will be provided from the NEMD plug to high ground. This levee will prevent American River backwater from reaching the upper reaches of the NEMD. The pumping plant described under Interior Drainage will be place at the NEMD plug. This new levee will require ramping of Ascot Avenue over the levee. Also a flood gate structure will be placed in the levee for the existing railroad. This structure will remain open until flood flows approach the top of the railroad tracks. By using this structure, the railroad tracks will not have to be ramped over the new levee. This same type of structure currently exists in the south levee of Dry Creek and the north and south levees of Arcade Creek. Operation instructions for these structures are contained in the Sacramento emergency procedures. The structure will only have to be closed during infrequent flood events.

TABLE N-1-5

CONSTRUCTION COST ONE-THIRD NATOMAS CROSS LEVEE
OCTOBER 1989 PRICES

ITEM	DESCRIPTION	COSTS
01	LANDS	37,500,000
02	RELOCATIONS	
	DEL PASO ROAD	370,000
	INTERSTATE 5	2,240,000
	DRAINAGE & IRRIGATION CANALS	1,050,000
	CONTINGENCIES 25%	910,000
	SUBTOTAL	4,570,000
11	LEVEES	11,420,000
	CONTINGENCIES 25%	2,840,000
	SUBTOTAL	14,260,000
30	E&D	2,260,000
31	S&A	1,510,000
	TOTAL	60,100,000

CONSTRUCTION COST TWO-THIRD NATOMAS CROSS LEVEE
OCTOBER 1989 PRICES

ITEM	DESCRIPTION	COSTS
01	LANDS	1,700,000
02	RELOCATIONS	
	POWERLINE ROAD	120,000
	STATE HIGHWAY 99	1,800,000
	DRAINAGE & IRRIGATION CANALS	1,000,000
	CONTINGENCIES 25%	750,000
	SUBTOTAL	3,670,000
11	LEVEES	11,180,000
	CONTINGENCIES 25%	2,830,000
	SUBTOTAL	14,010,000
30	E&D	2,120,000
31	S&A	1,400,000
	TOTAL	22,900,000

TABLE N-1-6

LEVEES - DESCRIPTION OF PLAN MODIFICATIONS
100-YEAR, 115,000 CFS ALTERNATIVE

REACH	LENGTH OF REACH (MI)	MODIFI-CATION REACH (MI)	MAXIMUM LEVEE HEIGHT INCREASE (FT)	ADDITIONAL RIGHT OF WAY (AC)	RELOCATIONS
NEMD EAST LEVEE					
AM R TO ARCADE CR	1.5	NO WORK REQUIRED			
ARCADE CR TO DRY CR	2.6	1.3	1.0	1.4	
NEMD WEST LEVEE					
EL CAMINO RD TO MAIN ST	3.2	2.5	2.0	0.5	REPLACE MAIN AV BRIDGE
MAIN ST TO NEMD PLUG	0.7	NO WORK REQUIRED			
RIEGO RD TO SANKEY RD	2.0	0.6	1.1	1.1	REPLACE ROAD ON LEVEE, RAMP SANKEY RD
RIEGO RD TO SANKEY RD	10,600 FT OF NEW 50 FT WIDE TRAINING CHANNEL			23.9	PROVIDE RR OPENING
ARCADE CREEK					
SOUTH LEVEE	2.1	0.2	1.3	0.4	1300 FT FENCE, 600 FT POWERLINE
NORTH LEVEE	1.9	0.4	3.1	0.7	1000 FT FENCE
DRY CREEK					
SOUTH LEVEE	1.3	0.1	0.5	0.2	
SOUTH NEW EXTENSION LEVEE	NA	0.3	3.1	1.8	
NEW NORTH LEVEE	NA	0.9	8.1	5.6	RAMP ASCOT AV, 200 FT FENCE STOP LOG STRUCTURE AT RR
NEMD PLUG		NEW 700 CFS PUMP STATION			

TABLE N-1-6 (Cont.)

REACH	LENGTH OF REACH (MI)	MODIFICATION REACH (MI)	MAXIMUM LEVEE HEIGHT INCREASE (FT)	ADDITIONAL RIGHT OF WAY (AC)	RELOCATIONS
PLEASANT GROVE CREEK CANAL	3.3	0.1	1.6	1.0	REPLACE ROAD ON LEVEE RAMP HOWSLEY RD, 1000 FT POWER AND TELEPHONE LINES
NATOMAS CROSS CANAL SOUTH LEVEE	5.4	1.1	1.1	0.0	RAMP HWY 99
NATOMAS DETENTION BASIN	NA	2.2	17.0	29.3	ADDITIONAL 279.2 ACRES OF FLOWAGE EASEMENT REQUIRED

The Pleasant Grove Creek Canal levee will be raised at Fifield Road and at Howsley Road. This will require replacing some of the public road on this levee and providing a ramp at Howsley Road.

Reaches of the south levee of the NCC will be raised. All of this work will take place on top of the levee. This work will insure that the south levee of the NCC does not fail for this level of protection. There is a risk that this work could cause a hydraulic impact for large floods. To mitigate for this impact several measures were investigated before the final was chosen.

Natomas Detention Basin - The draft report included the lengthening of Fremont Weir as the hydraulic mitigation measure for the raising of the south levee of the NCC. Initial analysis in early 1990 using the failure criteria established at that time, indicated that the stages in the NCC would be high enough to cause a break in the south levee of the NCC during severe floods. Should this break be prevented by levee raising, our hydrologic model showed that the impact to existing flood conditions in the NCC and the Pleasant Grove area would be an increase in these elevations of approximately 0.4 foot. Computation of this difference is difficult. The first measure proposed for mitigation of this increase was to buy flood easements in the Pleasant Grove area. At a public meeting in Pleasant Grove, the residents objected strongly to this measure. Because of these objections, another measure was formulated. Lengthening Fremont Weir was evaluated, to see if this would lower elevations in the Sacramento River enough to offset the impact of eliminating the break in the south levee of the NCC. Our initial analysis indicated that this was the case and the tentatively selected measure for mitigation of the increased flood levels in the Pleasant Grove area became lengthening the Fremont Weir.

Concern over the effectiveness of lengthening the weir was expressed by the State. As a part of responding to comments and finalizing the Draft Feasibility Report, additional analysis was made of the Fremont Weir lengthening measure. This additional investigation determined that the assumption of weir action for the Fremont Weir is not appropriate for the large flows being investigated. Hydraulic control for these flows is located downstream in the Yolo Bypass and modifications to the weir or to the bypass just downstream from the weir will not impact elevations. Therefore, based on the recent hydraulic analysis, the proposed lengthening of the weir and widening of the bypass as a hydraulic mitigation measure has been determined to not be effective.

Work is necessary on the south levee of the NCC and for the levee along the Pleasant Grove Creek Canal as part of all of the alternatives. This work will be to insure consistent design project conditions for the Natomas area. That is to insure three feet of freeboard for the design flood. Currently the west levee of the Sacramento River and the north levee of the NCC are lower than the levees on the opposite side of the river. These lower levees have a greater probability to fail before the levees which surround the Natomas area. These earlier failures influence elevations in the Cross Canal by making them lower.

However, at rare floods, there is a risk of some hydraulic impact to the Pleasant Grove Area due to uncertainties in these rare flood events. Based on prior runs, this impact has been conservatively

estimated at 0.4 feet. This impact occurs to an already existing and remaining flood plain of an average of 5 feet. For floods up to the 200-year event, no levee failure is anticipated according to Corps criteria, however a policy decision has been made to provide mitigation for the rare events impact.

To remove this impact, the alternatives have been revised to include a detention basin in the northeast corner of the Natomas area to store the increased flood elevation, see Plate 1. This area is currently used for rice farming and would be leveed to prevent the stored water from flooding the Natomas area. A 0.4 foot increase over the Pleasant Grove area is approximately 2,000 acre feet of volume. As a contingency, this amount has been increased to 3,000 acre feet. The detention basin has been sized to store 3,000 acre feet at 10 feet of depth and therefore covers approximately 300 acres. Water will flow into the detention basin through 6 - 8x8 concrete box culverts placed in the Pleasant Grove Canal levee. These culverts are approximately 100 feet long and will be controlled by sluice gates. The culverts have been sized to drain the 3,000 acre feet over a 12 hour time frame at the peak of the flood. A 48 inch culvert will be placed in the south levee of the detention basin to drain interior flows from the detention basin. This culvert will flow into the existing Natomas drainage system.

The detention basin will not be utilized until elevations in the Pleasant Grove Area reach a predetermined flood elevation, approximately 40.5 NGVD. At that point the culverts will be fully opened and the detention basin allowed to fill. This operating scheme allows the detention basin to skim the impacting storage off the peak of the flood. The culverts will remain open until the flood passes. Most of the water stored in the detention basin will flow out of the basin through the culverts and into the Pleasant Grove Canal as the flood recedes. The gates will be closed when the flood elevation recedes to 30 NGVD. Any remaining stored water will be released into the existing Natomas drainage system through the 48 inch interior drainage culvert located in the south levee of the detention basin. The area within the detention basin can still be farmed for most of the time. Flood operations will have an insignificant impact on farming operations.

Environmental impacts for the detention basin will be similar to but less than those identified for the Fremont Weir lengthening. This refinement in hydraulic mitigation measures will also require a new location for the fish and wildlife mitigation area for the levee work. These differences are discussed in more detail in the EIS.

All alternatives which include raising the south levee of the NCC will include this same size and type of detention basin as a hydraulic mitigation measure.

100-Year, 130,000 CFS Objective Release

This alternative is very similar to the preceding one. However, the increased objective release in the American River will cause higher water surface elevations in the lower portion of the NEMD and will require more levee work along Arcade and Dry Creeks. In addition, there will be work along the American River, the Sacramento Bypass, and in the

lower reaches of the Yolo Bypass. Table N-1-7 describes the modification work required for this alternative and Plate 1 can be used to locate work areas.

Work around the Natomas area will be the same as described for the 100-year, 115,000 cfs objective release alternative. As mentioned, work in the lower portion of the NEMD will be more extensive.

Increasing the objective release from Folsom Dam to 130,000 cfs would enable flood control storage upstream to provide a greater degree of flood protection. However, an increase in the objective release from Folsom Dam also requires strengthening and raising of the American River Levees. A stability analysis, Appendix M, Chapter 2, established that while these levees were stable for 115,000 cfs, releases above this would require, in addition to raising the levees, stability work such as drainage berms, toe drains, or slurry cutoff walls. Toe Drains were used where work area was available at the toe of the existing levee. In those reaches where development had occurred up to the toe, slurry cutoff walls were designed. There would also have to be extensive erosion protection, in the form of riprap, along the banks and levees for these alternatives. The erosion protection analysis is given in Appendix M, Chapter 4. Plate 2 shows the modifications required on the American River Levees for the 130,000 cfs objective release and Table N-1-8 summarizes the work required for this measure. No additional real estate would be needed along the American River levees because widening would be done on the waterside of the levees in the already existing floodway. While this measure has been designed and cost estimated, it is not considered as technically viable as a flood control measure for the American River levee system as providing upstream storage. High levees are inherently less safe than other flood control measures such as upstream storage, channels, or bypasses. While every attempt is made to include all work needed to insure the safety of the levee system, 23 miles of levee must withstand the increased forces and velocities of a higher objective release. It takes only one weak section in these long levee reaches to create a catastrophic scenario. The American River levees barely survived the 1986 flood which had a discharge of 130,000 cfs. Levee rehabilitation was necessary in several areas after this flood. Had this discharge lasted longer than the 24 hours it did, there would have most probably been a breach in some area. This type of flood control measure becomes particularly dangerous if the objective release represents a relatively frequent event of flooding such as the 100-year flood. Historically, the Corps of Engineers would only utilize high levees in an urban area if they provided protection for at least the Standard Project Flood (SPF), which represents a rare event. If these measures were to represent flood control for some flood less than the SPF, the Corps should not responsibly recommend them as a prudent flood control alternative.

With the increase in the objective release, the Folsom Dam Spillway will be modified. This modification entails lowering the spillway crest. It includes removing the eight existing 42 foot wide by 50 foot high tainter gates, lowering the spillway crest by fifteen feet, installing eight new 42 foot wide by 65 foot high tainter gates, and lengthening the stilling basin by fifty feet.

TABLE N-1-7

LEVEES - DESCRIPTION OF PLAN MODIFICATIONS
100-YEAR, 130,000 CFS ALTERNATIVE

REACH	LENGTH OF REACH (MI)	MODIFI-CATION REACH (MI)	MAXIMUM ADDITIONAL LEVEE HEIGHT INCREASE (FT)	MAXIMUM ADDITIONAL RIGHT OF WAY (AC)	RELOCATIONS
NEMD EAST LEVEE					
AM R TO ARCADE CR	1.5	NO WORK REQUIRED			
ARCADE CR TO DRY CR	2.6	2.5	1.9	3.0	
NEMD WEST LEVEE					
EL CAMINO RD TO MAIN ST	3.2	3.0	2.7	1.5	REPLACE MAIN AV BRIDGE
MAIN ST TO NEMD PLUG	0.7	NO WORK REQUIRED			
RIEGO RD TO SANKEY RD	2.0	0.6	1.1	1.1	REPLACE ROAD ON LEVEE, RAMP SANKEY RD
RIEGO RD TO SANKEY RD	10,600 FT OF NEW 50 FT WIDE TRAINING CHANNEL			23.9	PROVIDE RR OPENING
ARCADE CREEK					
SOUTH LEVEE	2.1	0.5	2.1	0.6	1300 FT FENCE, 600 FT POWERLINE
NORTH LEVEE	1.9	1.7	3.9	1.8	1000 FT FENCE
DRY CREEK					
SOUTH LEVEE	1.3	0.3	0.9	0.3	
SOUTH NEW EXTENSION LEVEE	NA	0.4	3.5	1.9	
NEW NORTH LEVEE	NA	0.9	8.5	5.8	RAMP ASCOT AV, 200 FT FENCE STOP LOG STRUCTURE AT RR
NEMD PLUG		NEW 700 CFS PUMP STATION			

TABLE N-1-7 (Cont.)

REACH	LENGTH OF REACH (MI)	MODIFI-CATION REACH (MI)	MAXIMUM LEVEE HEIGHT INCREASE (FT)	ADDITIONAL RIGHT OF WAY (AC)	RELOCATIONS
PLEASANT GROVE CREEK CANAL	3.3	0.1	1.6	1.0	REPLACE ROAD ON LEVEE RAMP HOWSLEY RD, 1000 FT POWER AND TELEPHONE LINES
NATOMAS CROSS CANAL SOUTH LEVEE	5.4	1.1	1.1	0.0	RAMP HWY 99
NATOMAS DETENTION BASIN	NA	2.2	17.0	29.3	ADDITIONAL 279.2 ACRES OF FLOWAGE EASEMENT REQUIRED
AMERICAN RIVER	SEE TABLE N-1-8				
FOLSOM SPILLWAY	LOWER SPILLWAY CREST 15 FEET				
SACRAMENTO BYPASS NORTH LEVEE NEW OFFSET LEVEE	1.8	1.8	26.0	110.0	INCLUDES NEW FLOWAGE EASEMENTS
SOUTH LEVEE	1.1	0.6	2.0	0.5	
SACRAMENTO WEIR	LENGTHEN WEIR TO THE NORTH BY 500 FT				LENGTHEN HIGHWAY BRIDGE 500 FEET LENGTHEN RR BRIDGE 500 FEET
YOLO BYPASS US AND DS OF SACRAMENTO BYPASS	RAISE NUMEROUS LOW SPOTS FOR HYDRAULIC MITIGATION				

TABLE N-1-8

LOWER AMERICAN RIVER
MODIFICATIONS REQUIRED FOR 130,000 CFS OBJECTIVE RELEASE

LEFT LEVEE

Station 85+00 to 95+00	Slurry Wall in Levee
Station 120+00 to 130+00	Toe Drain
Station 220+00 to 245+00	Slurry Wall in Levee
Station 550+00 to 600+00	New Levee, Maximum Height 5 Feet
Station 545+00	New Pump Station at Mayhew Drain

RIPRAP REQUIREMENTS

Station 0+00 to 13+00	Riprap Levee
Station 13+00 to 49+00	Riprap Levee and Bank
Station 96+00 to 123+00	Riprap Levee and Bank
Station 187+00 to 204+00	Riprap Levee and Bank
Station 216+00 to 251+00	Riprap Levee and Bank
Station 300+00 to 353+00	Riprap Levee and Bank
Station 353+00 to 393+00	Riprap Levee
Station 462+00 to 503+00	Riprap Levee

RIGHT LEVEE

Station 230+00 to 240+00	Toe Drain, Land Side Toe
Station 360+00 to 370+00	Toe Drain, Land Side Toe

NOTE: Right Levee is Not Raised for 130,000 cfs Objective Release

RIPRAP REQUIREMENTS

R.M. 0.0 to 0.3	Riprap Bank
R.M. 1.9 to 2.3	Riprap Bank
Station 8+00 to 19+00	Riprap Levee
R.M. 3.3 to 4.1	Riprap Bank
Station 169+00 to 199+00	Riprap Levee
Station 221+00 to 354+00	Riprap Levee
Station 370+00 to 378+00	Riprap Levee

SUMMARY

0.7 Miles Slurry Wall
0.6 Miles Toe Drain
0.9 Miles of New Levee
1.5 Miles of Riprap on Bank
5.3 Miles of Riprap on Levee
3.2 Miles of Riprap on Bank and Levee

Raise Right Trestle on Union Pacific Railroad - River Mile 2.5
Relocate 600 Feet of Park Road

The increased objective release flows mean increased flows in the Sacramento River and Yolo Bypass for a design event. For this alternative, it was assumed the Sacramento Weir and Bypass would be widened enough to divert the excess release above 115,000 cfs through the Sacramento Bypass and into the Yolo Bypass. Analysis using the DWOPER program indicated that widening the weir and bypass an additional 500 feet for the 130,000 cfs objective release would draw the increased objective flow up the Sacramento River and over the Sacramento weir. The widening requires new setback levees on the north side of the bypass and extension of the weir, a roadway bridge, and a railroad bridge. Lengthening the weir prevents any increase in flows in the Sacramento River below the American River and there would be no need for hydraulic mitigation downstream along the Sacramento River. The increased flows into the Yolo Bypass however would require extensive hydraulic mitigation along the Yolo Bypass both upstream and downstream of the Sacramento Bypass.

150-Year, 180,000 CFS Objective Release

All measures included in the 100-year, 130,000 cfs alternative are included in this one. However, because the objective release is much higher and the level of protection is higher, work effort will be much more. This alternative requires more relocations than the previous and the modification to the Sacramento Weir will be more extensive. Table N-1-9 lists the modifications required for this alternative. The following discusses the major differences between the 130,000 cfs objective release alternative and this alternative.

Because the levee raising along the NEMD will be much higher, the encroachment into the canal of the east levee is much greater, on the average of 14 feet. This amount of encroachment is not acceptable and for this alternative the levee raising will be to the landside for the east levee. This requires that the Western Pacific Railroad, which runs along the landside toe of this levee, be relocated. In addition, the higher objective release will require that the El Camino and Norwood Avenue Bridges be replaced.

Work along the American River is much more extensive. Plate 3 shows the location and type of effort required to stabilize the American River levees and raise them adequately to convey an objective release of 180,000 cfs. Table N-1-10 summarizes this work. The discussion on whether this is as prudent a viable measure as upstream storage will not be repeated. However, the reluctance to recommend an alternative which includes increased objective releases in the American River is even stronger for this alternative.

The Sacramento Weir will be lengthened an additional 3600 feet. Lengthening of the Sacramento Weir will be an extremely difficult undertaking due to the railroad and public road which cross it. In addition, diverting the increased objective release into the Yolo Bypass will require extensive levee raising on both sides of the Yolo Bypass. This raising will extend almost to Rio Vista on the east side. Downstream land owners in the Yolo Bypass will probably object extensively to any increased flows in the Yolo Bypass.

TABLE N-1-9

LEVEES - DESCRIPTION OF PLAN MODIFICATIONS
150-YEAR, 180,000 CFS ALTERNATIVE

REACH	LENGTH OF REACH (MI)	MODIFICATION REACH (MI)	MAXIMUM ADDITIONAL LEVEE HEIGHT INCREASE (FT)	MAXIMUM ADDITIONAL RIGHT OF WAY (AC)	RELOCATIONS
NEMD EAST LEVEE					
AM R TO ARCADE CR	1.5	1.1	1.9	1.6	REPLACE EL CAMINO AV BRIDGE
ARCADE CR TO DRY CR	2.6	2.6	3.4	5.5	RELOCATE WESTERN PACIFIC RR
NEMD WEST LEVEE					
EL CAMINO RD TO MAIN ST	3.2	3.2	4.2	5.0	REPLACE MAIN AV BRIDGE
MAIN ST TO NEMD PLUG	0.7	0.7	1.0	2.1	REPLACE ROAD ON LEVEE
RIEGO RD TO SANKEY RD	2.0	0.6	1.1	1.1	REPLACE ROAD ON LEVEE, RAMP SANKEY RD
RIEGO RD TO SANKEY RD	10,600 FT OF NEW 80 FT WIDE TRAINING CHANNEL			23.9	PROVIDE RR OPENING
ARCADE CREEK					
SOUTH LEVEE	2.1	0.7	2.6	0.8	1300 FT FENCE, 600 FT POWERLINE
NORTH LEVEE	1.9	1.9	4.3	4.1	1000 FT FENCE, REPLACE NORWOOD AV BRIDGE
DRY CREEK					
SOUTH LEVEE	1.3	0.6	1.9	0.9	
SOUTH NEW EXTENSION LEVEE	NA	0.4	6.0	2.1	
NEW NORTH LEVEE	NA	0.9	9.9	6.5	RAMP ASCOT AV, 200 FT FENCE STOP LCG STRUCTURE AT RR
NEMD PLUG	NEW 700 CFS PUMP STATION				

TABLE N-1-9 (Cont.)

REACH	LENGTH OF REACH (MI)	MODIFI-CATION REACH (MI)	MAXIMUM LEVEE HEIGHT INCREASE (FT)	ADDITIONAL RIGHT OF WAY (AC)	RELOCATIONS
PLEASANT GROVE CREEK CANAL	3.3	0.1	1.7	1.0	REPLACE ROAD ON LEVEE RAMP HOWSLEY RD, 1000 FT POWER AND TELEPHONE LINES
NATOMAS CROSS CANAL SOUTH LEVEE	5.4	1.1	1.3	0.0	RAMP HWY 99
NATOMAS DETENTION BASIN	NA	2.2	17.0	29.3	ADDITIONAL 279.2 ACRES OF FLOWAGE EASEMENT REQUIRED
AMERICAN RIVER	SEE TABLE N-1-10				
FOLSOM SPILLWAY	LOWER SPILLWAY CREST 15 FEET				
SACRAMENTO BYPASS					
NORTH LEVEE					
NEW OFFSET LEVEE	1.8	1.8	26.0	760.0	INCLUDES NEW FLOWAGE EASEMENTS
SOUTH LEVEE	1.1	0.6	2.0	0.5	
SACRAMENTO WEIR	LENGTHEN WEIR TO THE NORTH BY 3600 FT				LENGTHEN HIGHWAY BRIDGE 3600 FT LENGTHEN RR BRIDGE 3600 FT
YOLO BYPASS					
US AND DS OF SACRAMENTO BYPASS	EXTENSIVE RAISING OF LEVEES FOR HYDRAULIC MITIGATION				

TABLE N-1-10

LOWER AMERICAN RIVER
MODIFICATIONS REQUIRED FOR 180,000 CFS OBJECTIVE RELEASE

LEFT LEVEE

Station 42+00 to 65+00	Toe Drain
Station 85+00 to 105+00	Slurry Wall in Levee
Station 120+00 to 130+00	Toe Drain
Station 192+00 to 200+00	Toe Drain
Station 202+00 to 315+00	Slurry Wall in Levee
Station 315+00 to 402+00	Toe Drain, Land Side Toe
Station 402+00 to 485+00	Slurry Wall in Levee
Station 346+00 to 550+00	Raise Existing Levee, Water Side, Maximum Levee height Increase 3.5 Feet
Station 550+00 to 615+00	New Levee, Maximum Height 9 Feet
Station 660+00 to 675+00	Raise Existing Levee, Water Side, Maximum Levee Height Increase 2 Feet
Station 545+00	New Pump Station at Mayhew Drain

RIPRAP REQUIREMENTS
SAME AS 130,000 CFS OBJECTIVE RELEASES

Station 0+00 to 13+00	Riprap Levee
Station 13+00 to 49+00	Riprap Levee and Bank
Station 96+00 to 123+00	Riprap Levee and Bank
Station 187+00 to 204+00	Riprap Levee and Bank
Station 216+00 to 251+00	Riprap Levee and Bank
Station 300+00 to 353+00	Riprap Levee and Bank
Station 353+00 to 393+00	Riprap Levee
Station 462+00 to 503+00	Riprap Levee

RIGHT LEVEE

Station 160+00 to 445+00	Toe Drain, Land Side Toe
Station 150+00 to 541+00	Raise Levee, Water Side, Maximum Levee Height Increase 4 Feet

RIPRAP REQUIREMENTS
SAME AS 130,000 CFS OBJECTIVE RELEASES

R.M. 0.0 to 0.3	Riprap Bank
R.M. 1.9 to 2.3	Riprap Bank
Station 8+00 to 19+00	Riprap Levee
R.M. 3.3 to 4.1	Riprap Bank
Station 169+00 to 199+00	Riprap Levee
Station 221+00 to 354+00	Riprap Levee
Station 370+00 to 378+00	Riprap Levee

TABLE N-1-10 (CONT.)

LOWER AMERICAN RIVER
MODIFICATIONS REQUIRED FOR 180,000 CFS OBJECTIVE RELEASE

SUMMARY

4.1 Miles Slurry Wall
7.8 Miles Toe Drain
1.0 Miles New Levee
11.4 Miles Levee Raising
1.5 Miles of Riprap on Bank
5.3 Miles of Riprap on Levee
3.2 Miles of Riprap on Bank and Levee

Raise Right Trestle of Union Pacific Railroad - River Mile 2.5
Raise Right Side of H-Street Bridge - River Mile 6.4
Replace Howe Avenue Bridge - River Mile 7.8
Replace 2400 Feet of Park Road
Replace 21,800 Feet of Park Bike Trail
Replace 2,400 Feet of Fence

There are already many complaints about existing conditions and these downstream landowners have to maintain their levee elevations to stringent conditions so as not to impact adjacent land owners. Any increase in the frequency or amount of flooding will not be welcomed by these downstream interests.

200-Year, 115,000 CFS Objective Release

This alternative is very similar to the 100-year, 115,000 cfs objective release alternative. Levee work is slightly more because of the higher level of protection. However, 200-year elevations are only several tenths more than 100-year elevations. All discussions for the 100-year alternative apply here. Table N-1-11 lists the modifications required for this alternative.

400-Year, 115,000 CFS Objective Release

Again this alternative is very similar to the other two alternatives with the 115,000 cfs objective release. Minor increases in water surface elevations for the higher level of protection will create some additional levee work. Table N-1-12 lists the modifications required for this alternative.

Costs for the different levee alternatives investigated are shown on Table N-1-13.

TABLE N-1-11

LEVEES - DESCRIPTION OF PLAN MODIFICATIONS
200-YEAR, 115,000 CFS ALTERNATIVE

REACH	LENGTH OF REACH (MI)	MODIFI-CATION REACH (MI)	MAXIMUM LEVEE HEIGHT INCREASE (FT)	ADDITIONAL RIGHT OF WAY (AC)	RELOCATIONS
NEMD EAST LEVEE					
AM R TO ARCADE CR	1.5	NO WORK REQUIRED			
ARCADE CR TO DRY CR	2.6	1.3	1.0	1.4	
NEMD WEST LEVEE					
EL CAMINO RD TO MAIN ST	3.2	2.5	2.0	0.5	REPLACE MAIN AV BRIDGE
MAIN ST TO NEMD PLUG	0.7	NO WORK REQUIRED			
RIEGO RD TO SANKEY RD	2.0	0.6	1.1	1.1	REPLACE ROAD ON LEVEE, RAMP SANKEY RD
RIEGO RD TO SANKEY RD	10,600 FT OF NEW 100 FT WIDE TRAINING CHANNEL			23.9	PROVIDE RR OPENING
ARCADE CREEK					
SOUTH LEVEE	2.1	0.2	1.3	0.4	1300 FT FENCE, 600 FT POWERLINE
NORTH LEVEE	1.9	0.4	3.1	0.9	1000 FT FENCE
DRY CREEK					
SOUTH LEVEE	1.3	0.2	0.7	0.4	
SOUTH NEW EXTENSION LEVEE	NA	0.4	4.8	1.9	
NEW NORTH LEVEE	NA	0.9	8.3	5.6	RAMP ASCOT AV, 200 FT FENCE STOP LOG STRUCTURE AT RR
NEMD PLUG	NEW 700 CFS PUMP STATION				

TABLE N-1-11 (Cont.)

REACH	LENGTH OF REACH (MI)	MODIFI-CATION REACH (MI)	MAXIMUM LEVEE HEIGHT INCREASE (FT)	ADDITIONAL RIGHT OF WAY (AC)	RELOCATIONS
PLEASANT GROVE CREEK CANAL	3.3	0.1	1.8	1.0	REPLACE ROAD ON LEVEE RAMP HOWSLEY RD, 1000 FT POWER AND TELEPHONE LINES
NATOMAS CROSS CANAL SOUTH LEVEE	5.4	1.1	1.6	0.0	RAMP HWY 99
NATOMAS DETENTION BASIN	NA	2.2	17.0	29.3	ADDITIONAL 279.2 ACRES OF FLOWAGE EASEMENT REQUIRED

TABLE N-1-12

LEVEES - DESCRIPTION OF PLAN MODIFICATIONS
400-YEAR, 115,000 CFS ALTERNATIVE

REACH	LENGTH OF REACH (MI)	MODIFI-CATION REACH (MI)	MAXIMUM LEVEE HEIGHT INCREASE (FT)	ADDITIONAL RIGHT OF WAY (AC)	RELOCATIONS
NEMD EAST LEVEE					
AM R TO ARCADE CR	1.5	NO WORK REQUIRED			
ARCADE CR TO DRY CR	2.6	1.4	1.8	3.0	
NEMD WEST LEVEE					
EL CAMINO RD TO MAIN ST	3.2	2.6	2.8	1.9	REPLACE MAIN AV BRIDGE
MAIN ST TO NEMD PLUG	0.7	NO WORK REQUIRED			
RIEGO RD TO SANKEY RD	2.0	0.6	1.1	1.1	REPLACE ROAD ON LEVEE, RAMP SANKEY RD
RIEGO RD TO SANKEY RD	10,600 FT OF NEW 150 FT WIDE TRAINING CHANNEL			23.9	PROVIDE RR OPENING
ARCADE CREEK					
SOUTH LEVEE	2.1	0.2	1.5	0.6	1300 FT FENCE, 600 FT POWERLINE
NORTH LEVEE	1.9	0.5	3.3	1.0	1000 FT FENCE
DRY CREEK					
SOUTH LEVEE	1.3	0.2	0.9	0.4	
SOUTH NEW EXTENSION LEVEE	NA	0.5	5.0	1.9	
NEW NORTH LEVEE	NA	0.9	8.5	5.8	RAMP ASCOT AV, 200 FT FENCE STOP LOG STRUCTURE AT RR
NEMD PLUG	NEW 700 CFS PUMP STATION				

TABLE N-1-12 (Cont.)

REACH	LENGTH OF REACH (MI)	MODIFI-CATION REACH (MI)	MAXIMUM LEVEE HEIGHT INCREASE (FT)	ADDITIONAL RIGHT OF WAY (AC)	RELOCATIONS
PLEASANT GROVE CREEK CANAL	3.3	0.1	2.0	2.2	REPLACE ROAD ON LEVEE RAMP HOWSLEY RD, 1000 FT POWER AND TELEPHONE LINES
NATOMAS CROSS CANAL SOUTH LEVEE	5.4	1.1	1.8	0.0	RAMP HWY 99
NATOMAS DETENTION BASIN	NA	2.2	17.0	29.3	ADDITIONAL 279.2 ACRES OF FLOWAGE EASEMENT REQUIRED

TABLE N-1-13

CONSTRUCTION COSTS FOR LEVEE ALTERNATIVES

OCTOBER 1991 PRICES

	100-YEAR 115,000 CFS OBJECTIVE RELEASE	100-YEAR 130,000 CFS OBJECTIVE RELEASE	100-YEAR 145,000 CFS OBJECTIVE RELEASE	150-YEAR 180,000 CFS OBJECTIVE RELEASE	200-YEAR 115,000 CFS OBJECTIVE RELEASE	400-YEAR 115,000 CFS OBJECTIVE RELEASE
01 LANDS	6,400,000	13,700,000	17,100,000	25,300,000	8,500,000	10,200,000
02 RELOCATIONS	5,100,000	19,400,000	33,000,000	67,500,000	5,300,000	5,700,000
04 DAMS	0	40,600,000	0	40,600,000	0	0
09 CHANNELS AND CANALS	400,000	400,000	400,000	600,000	700,000	1,000,000
11 LEVEES	6,600,000	41,900,000	51,900,000	73,200,000	6,900,000	7,400,000
13 PUMPING PLANT	5,400,000	18,900,000	18,900,000	18,900,000	5,400,000	5,400,000
15 FLOODWAY CNTRL & DIVER STRUCTURES	0	3,000,000	8,200,000	20,400,000	0	0
30 ENG & DESIGN	2,200,000	14,900,000	13,500,000	26,600,000	2,200,000	2,400,000
31 SUPER & ADMIN	1,400,000	9,900,000	8,900,000	17,600,000	1,500,000	1,500,000
TOTAL FIRST COSTS	27,500,000	162,700,000	151,900,000	290,700,000	30,500,000	33,600,000

SELECTED PLAN

The selected plan for the American River Watershed is described in the following paragraphs. This is the plan recommended for construction. This plan includes additional flood control storage upstream on the American River at River Mile 20.1, described in Chapter 3 of this Appendix, and levee work in the Sacramento Area to provide 200-year protection. The objective release out of Folsom reservoir will remain at the current objective release of 115,000 cfs into the American River. Levees will have to contain this objective release with 200-year floods in the tributaries to the American River. Levees will be modified to also protect against the 200-year flood along the Sacramento River and its tributaries. This plan requires levee raising along the east and west levees of the Natomas East Main Drain, the south and north levees of Arcade Creek, the south levee of Dry Creek, the east levee of

the Pleasant Grove Creek Canal, and the south levee of the Natomas Cross Canal. New levees will be required as an extension of the south levee of Dry Creek and a north levee along Dry Creek. This new north levee will block the NEMD to prevent backwater from proceeding upstream. A 700 cfs pumping plant, described earlier in this chapter and in Chapter 2 of this appendix, with low flow sluices will be provided to handle flows behind this new levee. This pumping plant along with the storage in the NEMDC in this area is adequate to handle all interior flows as well as the diverted flows conveyed by the following described channel. A 100 foot wide trapezoidal grass lined channel will be constructed in the upper reach of the NEMD. This channel will be approximately 2 miles long and will provide hydraulic mitigation for flows prevented from entering the Natomas area after Sankey Road is raised. The Main Avenue Bridge will be replaced. A dip in the levee at Sankey road will be raised. In addition, Fifield and Howsley Roads will have to be raised where the levee is raised. The existing Fifield wooden bridge is in poor condition and floods submerge the existing structure. The proposed project will not increase flood heights so there is no flood impact from the project. The existing Fifield Bridge will have its west end raised approximately 0.8 feet and shims installed on the existing piers. The Howsley Bridge will also see no change in its existing flood impacts. The end of the bridge will be raised by adding asphalt concrete surfacing. Several refinements from the tentatively selected plan presented in the draft report to the selected plan in this final report have occurred. The change in hydraulic mitigation from lengthening Fremont Weir to the Natomas Detention Basin has already been discussed. The following discusses interior drainage in Natomas.

Natomas Area Interior Drainage Pumps

The tentatively selected plan in the Draft Feasibility Report included interior drainage pumping plants for the Natomas area. As mentioned earlier an extensive interior drainage system already exists for the existing Natomas levee system. The current interior drainage system has operated adequately in the past. The selected plan does not change these existing drainage patterns or intercept any new drains. However, the proposed project will allow development to occur which cannot occur without an increase in protection and this future development will increase interior runoff. Additional guidance since the draft report has directed that the impacts from this future development are considered indirect impacts and should be dealt with in a similar manner as future indirect fish and wildlife impacts. Accordingly, while there is a project cost for these facilities which handle this future increase in runoff, the construction and cost for these future facilities is entirely a responsibility of the non-Federal sponsor. Therefore these costs have been removed from the M-CACES for the selected plan. The plan and cost for handling this impact is given here as information only. The selected plan does not include these features in the M-CACES Base Line estimate.

The City of Sacramento has developed a plan to handle the increased runoff from the development of North Natomas. This plan has been detailed in the following reports:

Report on Drainage Study, North Natomas Area, Phase II, December 1987, Dewante and Stowell

Revised Draft Supplemental EIR for the North Natomas Community Drainage System, November 1989, Jones and Stokes

Final Supplemental EIR for the North Natomas Community Drainage System, June 1990, Jones and Stokes

Mitigation Monitoring Plan for the North Natomas Community Drainage System, Supplemental EIR, July 1990, Jones and Stokes

North Natomas is the only area in Natomas with an approved plan for development and is the future land use plan used for all evaluations of future flood protection benefits and indirect mitigation requirements for the project. The proposed drainage facilities include two canal systems which drain to two proposed new pumping stations. The canal systems are identified as the San Juan Canal and Del Paso Canal.

The San Juan Canal system has three primary canals for conveying storm runoff. The canals are grass lined. The East Drain portion will be 7,600 feet long and will consist of a trapezoidal canal with bottom widths ranging from 25 to 60 feet. The K Canal/West Drain will be 8,200 feet long and have bottom widths from 10 to 25 feet. The San Juan Canal will be 15,000 feet long with bottom widths ranging from 70 to 140 feet. This gives a total canal length for this portion of 30,800 feet. The San Juan Canal will drain to the San Juan Pumping Station which consists of 12 pumping units, 10 at 1000 HP powered by diesel engines and 2 - 500 HP electric pumps. This pump station has a pumping capacity of 2,200 cfs. No sump areas are proposed, the canals are large and do provide some sump storage. However, the canals are designed to convey the pumping capacity of the pump station.

The Del Paso Canal system also has three primary grass lined trapezoidal canals for drainage. The East Drain portion is 9,800 feet long with bottom widths ranging from 20 to 65 feet. This canal connects to the Elkhorn Canal, a 17,000 foot long canal with bottom widths of 110 to 140 feet. The Del Paso Canal is 15,000 feet long and has bottom widths ranging from 140 to 230 feet. This give a total canal length of 41,800 feet. This canal drains to the Del Paso Pumping Station. This station has a capacity of 3,700 cfs and consists of 12 pumping units, 10 at 2000 HP powered by diesel engines and 2 - 750 HP electric pumps. Again there are no specific sump areas and the canals are designed to convey the pump station capacity.

The total cost for this system is $122,000,000. These costs include lands, drainage canals, the two pumping stations, E&D, and S&I. Coordination with the City of Sacramento established that the City thinks the San Juan Plant would be constructed first in the 1994 time frame and the Del Paso Plant 10 years later (2004). This drainage

system will have to be built before or concurrently as development occurs in North Natomas.

Modified Reaches of the Selected Plan

Plates 4 through 11 show the extent of the levee work involved for the different reaches for the selected plan. Plate 1 shows the locations of the levees. The height increases for the levees in most cases are low enough so that most work will occur on top of the existing levee. Levee fill along the east levee of the NEMD will be waterside to avoid any impacts to the Western Pacific Railroad which runs along this levee. Table N-1-14 summarizes the levee modifications required for the selected plan.

Cost Estimate for the Selected Plan

The M-CACES Base Line cost estimate for the selected plan is provided in Chapter 4 of this appendix.

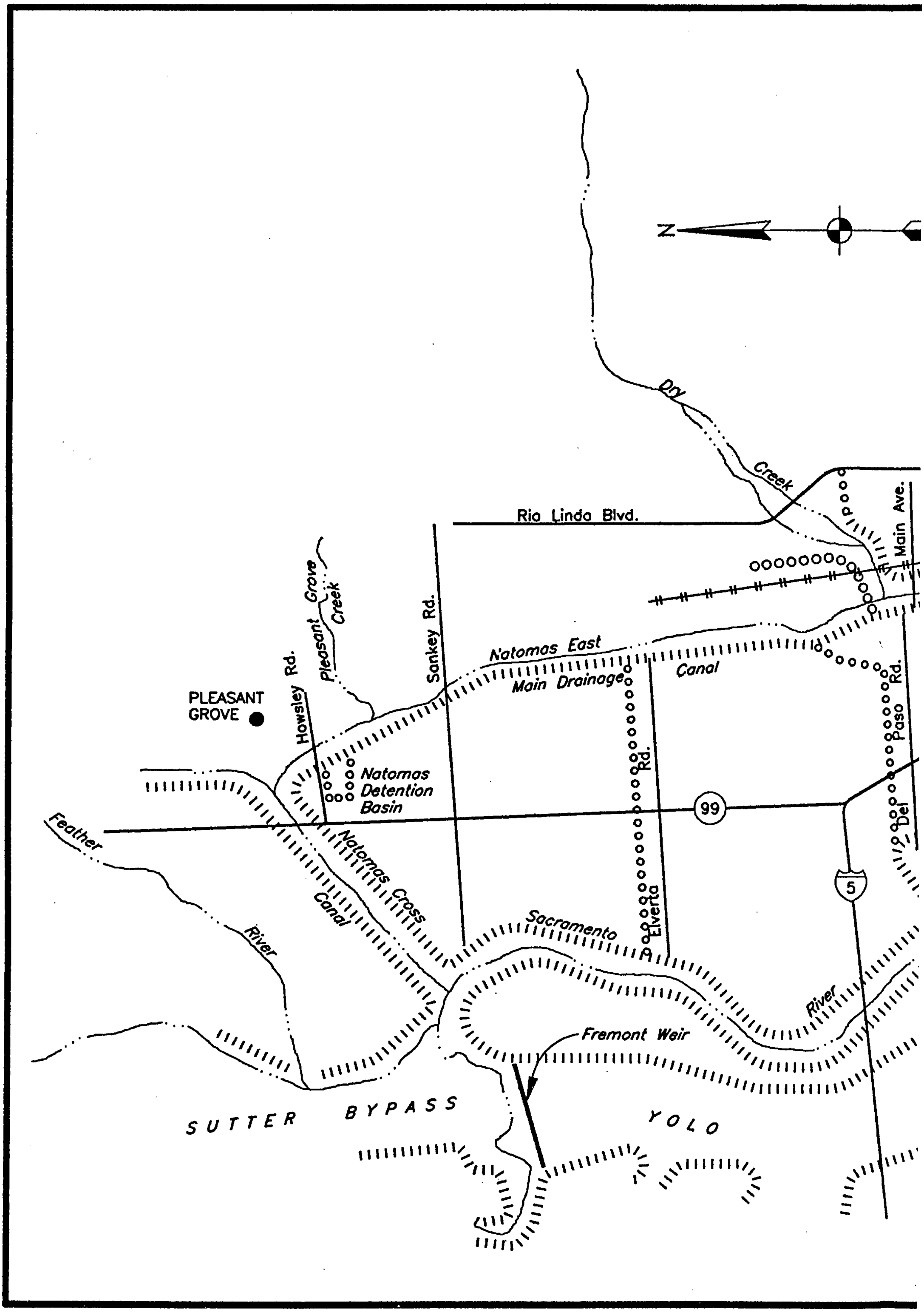
TABLE N-1-14

SELECTED PLAN
DESCRIPTION OF PLAN MODIFICATIONS

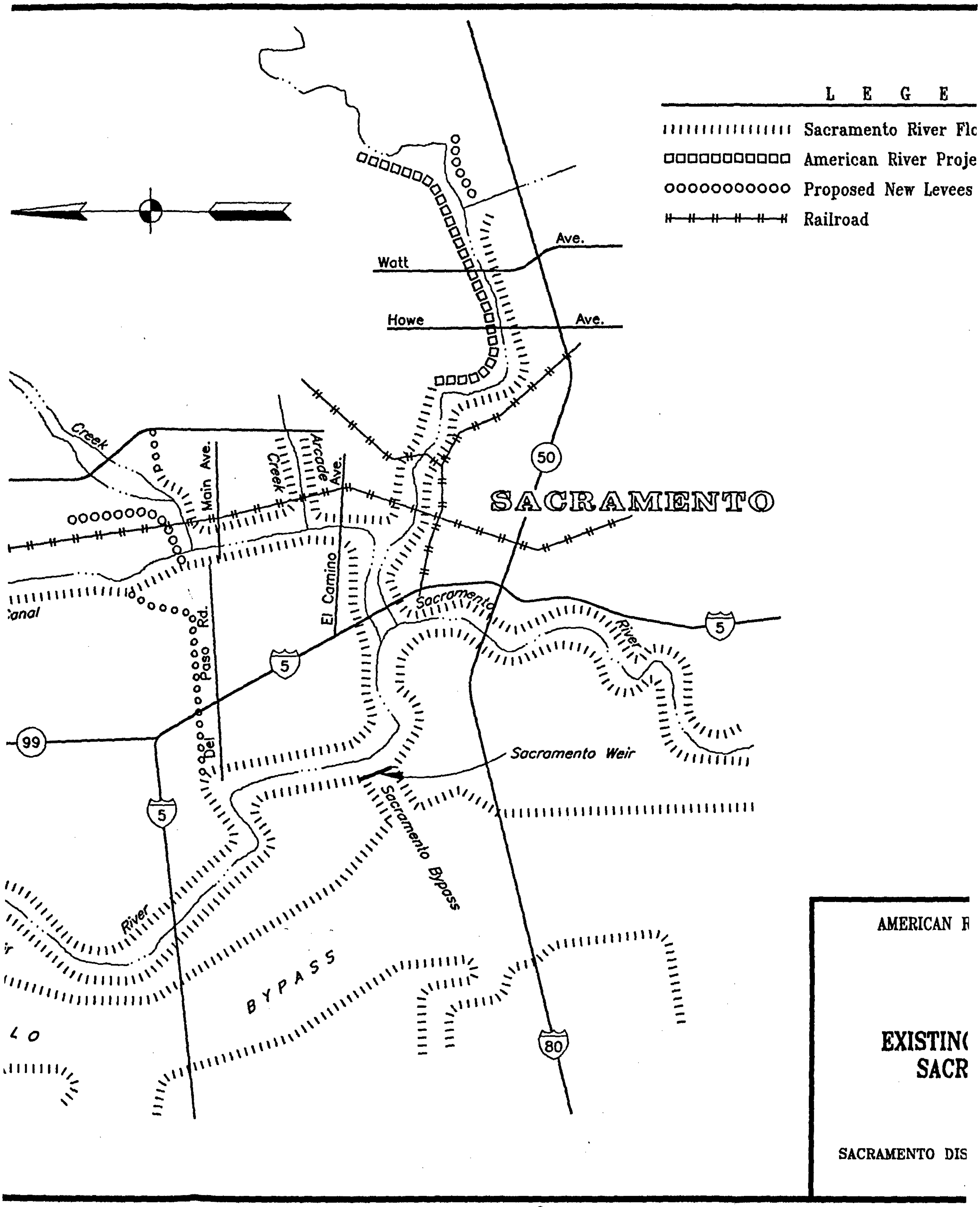
REACH	LENGTH OF REACH (MI)	MODIFI-CATION REACH (MI)	MAXIMUM LEVEE HEIGHT INCREASE (FT)	ADDITIONAL RIGHT OF WAY (AC)	RELOCATIONS
NEMD EAST LEVEE					
AM R TO ARCADE CR	1.5	NO WORK REQUIRED			
ARCADE CR TO DRY CR	2.6	1.4	1.8	3.5	
NEMD WEST LEVEE					
EL CAMINO RD TO MAIN ST	3.2	2.6	2.8	1.2	REPLACE MAIN AV BRIDGE
MAIN ST TO NEMD PLUG	0.7	NO WORK REQUIRED			
RIEGO RD TO SANKEY RD	2.0	0.6	1.1	2.6	REPLACE ROAD ON LEVEE, RAMP SANKEY RD
RIEGO RD TO SANKEY RD	10,600 FT OF NEW 100 FT WIDE TRAINING CHANNEL			24.0	PROVIDE RR OPENING
ARCADE CREEK					
SOUTH LEVEE	2.1	0.2	1.3	0.5	1,300 FT FENCE, 600 FT POWERLINE
NORTH LEVEE	1.9	0.5	3.1	1.7	1,400 FT FENCE
DRY CREEK					
SOUTH LEVEE	1.3	0.2	0.7	0.4	
SOUTH NEW EXTENSION LEVEE	NA	0.5	5.0	3.9	
NEW NORTH LEVEE	NA	0.9	8.5	9.5	RAMP ASCOT AV, 1,200 FT FENCE STOP LOG STRUCTURE AT RR
NEMD PLUG		NEW 700 CFS PUMP STATION			

TABLE N-14 (CONT.)

REACH	LENGTH OF REACH (MI)	MODIFICATION REACH (MI)	MAXIMUM LEVEE HEIGHT INCREASE (FT)	ADDITIONAL RIGHT OF WAY (AC)	RELOCATIONS
PLEASANT GROVE CREEK CANAL	3.3	0.1	2	0.0	REPLACE ROAD ON LEVEE RAMP HOWSLEY RD, 1200 FT POWER AND TELEPHONE LINES
NATOMAS CROSS CANAL SOUTH LEVEE	5.4	3.4	0.6	0.0	
NATOMAS DETENTION BASIN	NA	2.2	17.0	29.3	ADDITIONAL 279.2 ACRES OF FLOWAGE EASEMENT REQUIRED
BORROW SITE	125 ACRES OF TEMPORARY EASEMENT				
ENVIRONMENTAL MITIGATION	280 ACRES OF HABITAT RESTORATION				


N
Dry
Creek
Rio Linda Blvd.
Main Ave.
Pleasant Grove
Creek
Sankey Rd.
Howsley Rd.
PLEASANT
GROVE
Natomas East
Main Drainage
Canal
Natomas
Detention
Basin
Elverta
Rd.
Del
Paso
Rd.
99
5
Feather
River
Natomas Cross
Canal
Sacramento
River
Fremont Weir
SUTTER BYPASS
YOLO

1


L E G E
Sacramento River Flc
American River Proje
Proposed New Levees
Railroad
Watt Ave.
Howe Ave.
Creek
Main Ave.
Creek
Arcade Ave.
SACRAMENTO
50
Canal
El Camino
Del Paso Rd.
Sacramento River
5
99
Sacramento Weir
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River
BYPASS
LO
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AMERICAN R
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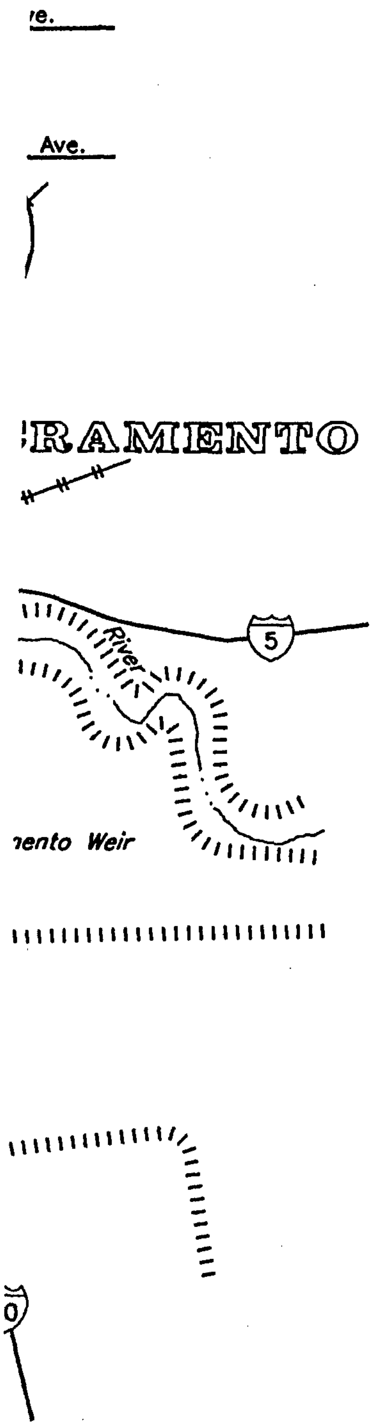
L E G E N D

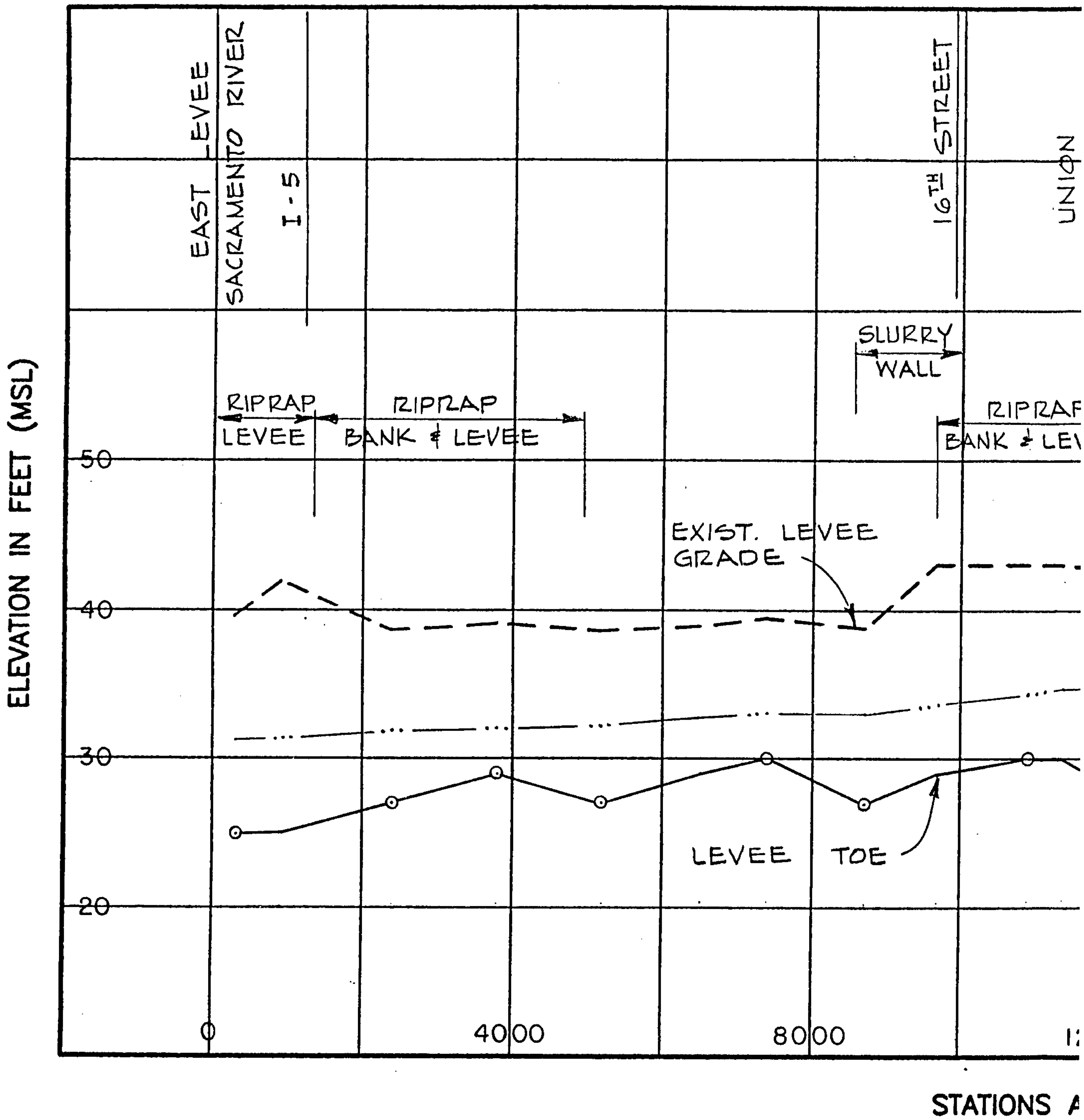
- Sacramento River Flood Control Project Levees
- American River Project Levees
- Proposed New Levees
- Railroad



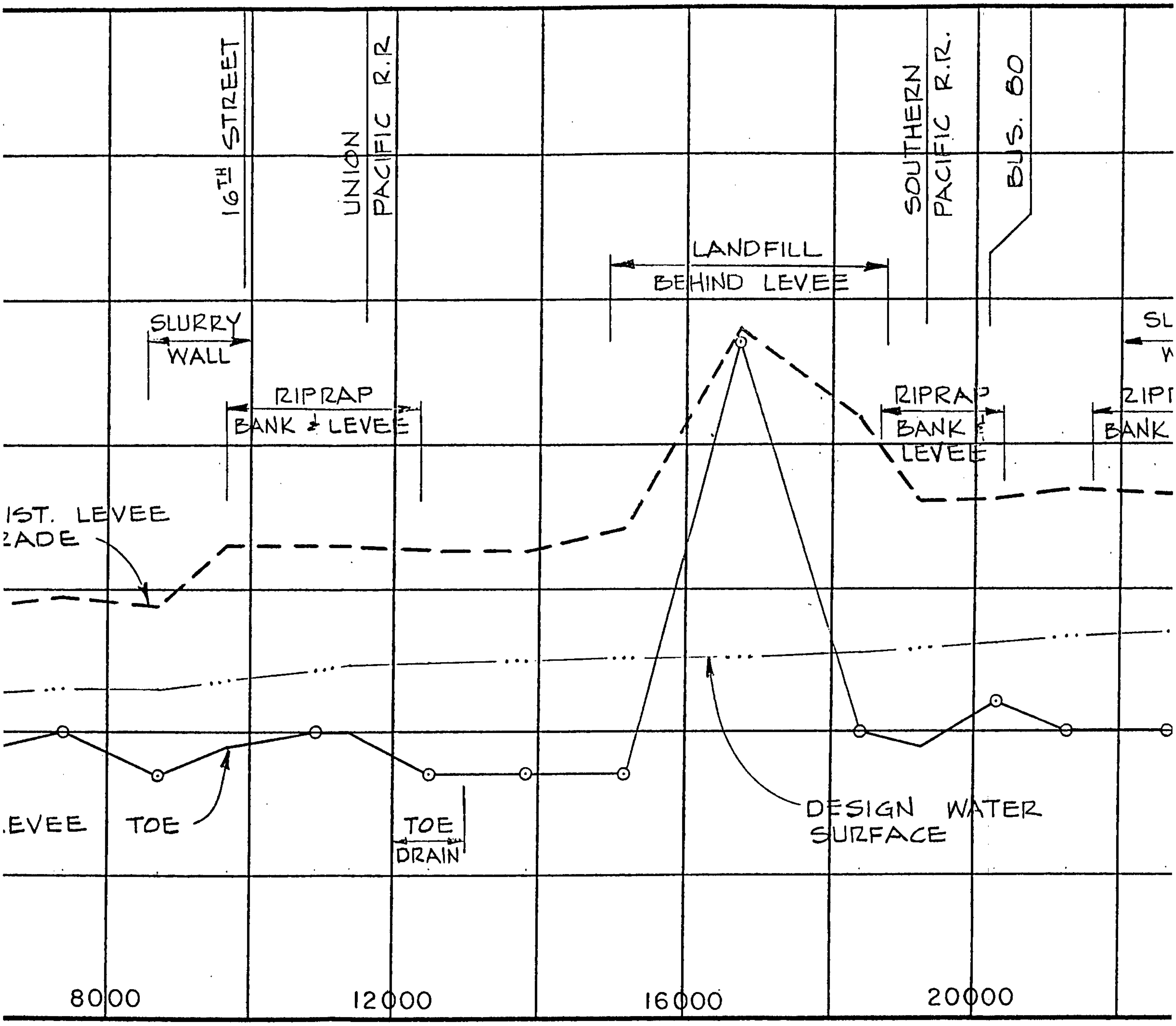
AMERICAN RIVER WATERSHED STUDY
CALIFORNIA

EXISTING LEVEES IN THE SACRAMENTO AREA

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

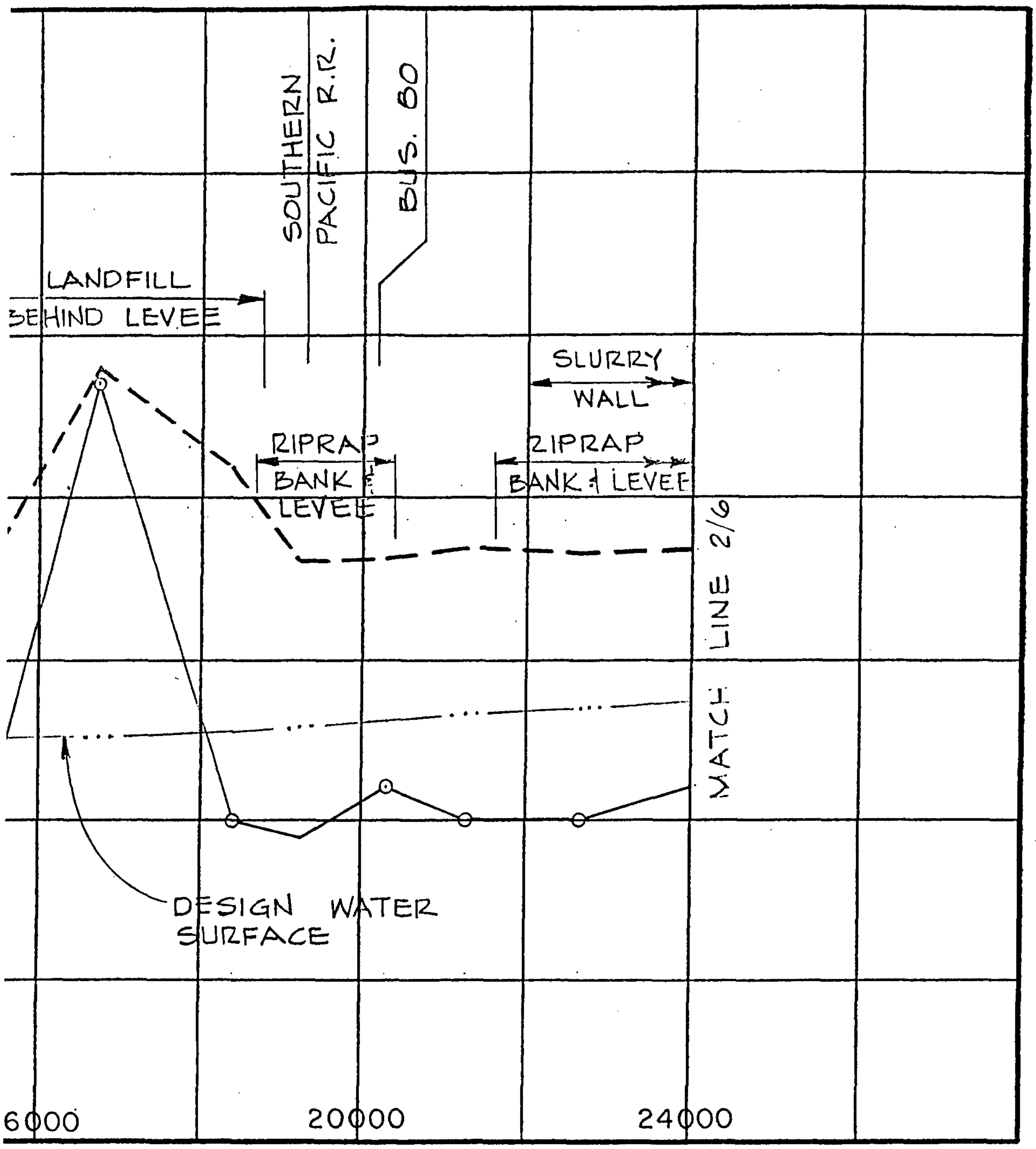

EAST LEVEE
SACRAMENTO RIVER
I-5
16TH STREET
UNION
SLURRY WALL
RIPRAP LEVEE
RIPRAP BANK & LEVEE
RIPRAP
BANK & LE
EXIST. LEVEE GRADE
LEVEE TOE
ELEVATION IN FEET (MSL)
50
40
30
20
0
4000
8000
STATIONS

1



STATIONS ALONG LEVEE CENTERLINE

AMERIC
W
LOW
SOUT
SACRAM



LINE

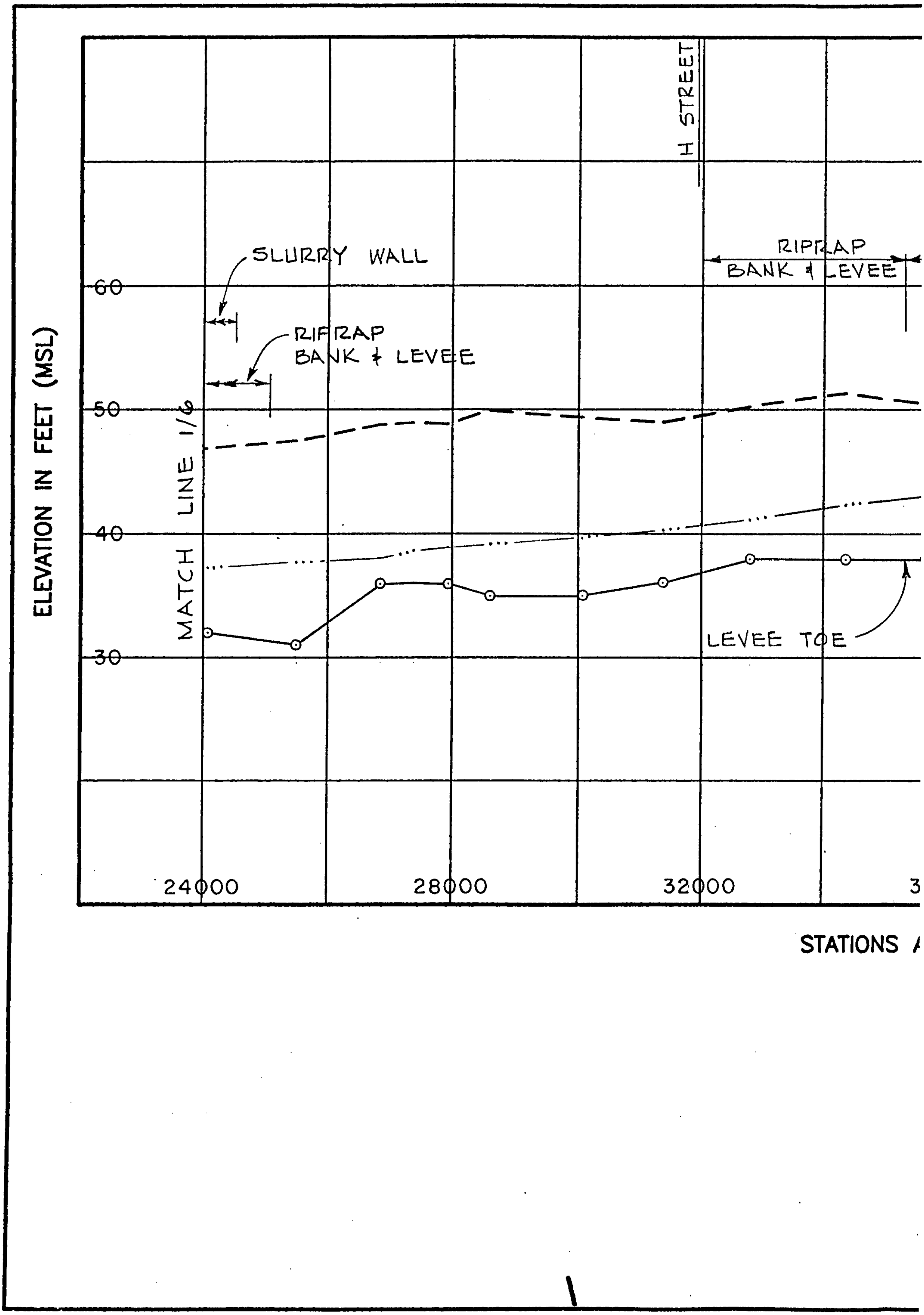
AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES

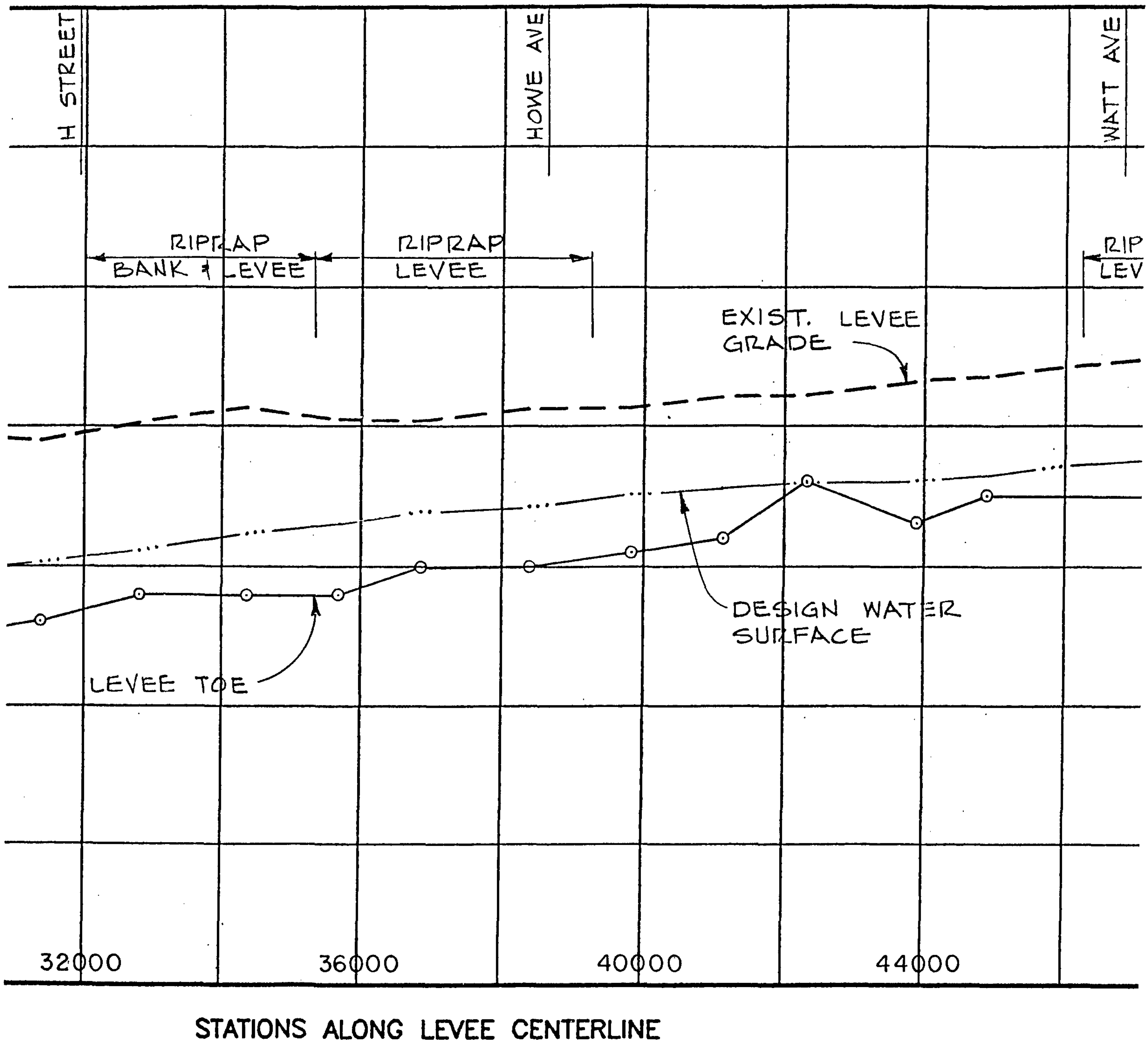
LOWER AMERICAN RIVER
SOUTH (LEFT) LEVEE PROFILE
130,000 CFS PLAN

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 2 1/6



H STREET
SLURRY WALL
RIPRAP BANK & LEVEE
RIPRAP BANK & LEVEE
MATCH LINE 1/6
LEVEE TOE
ELEVATION IN FEET (MSL)
60
50
40
30
24000
28000
32000
STATIONS



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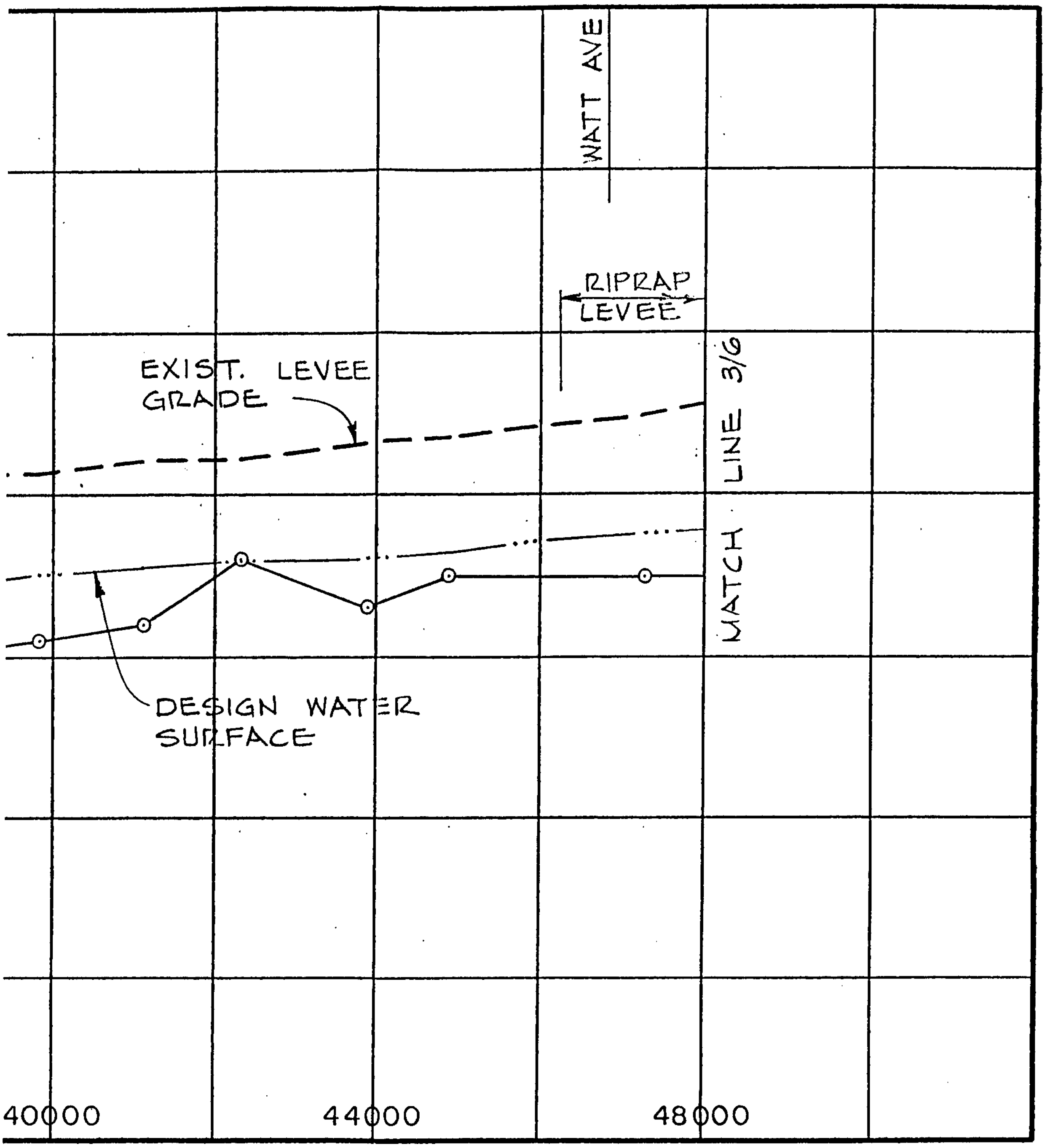
LOWE

SOUTH

13

SACRAMENT



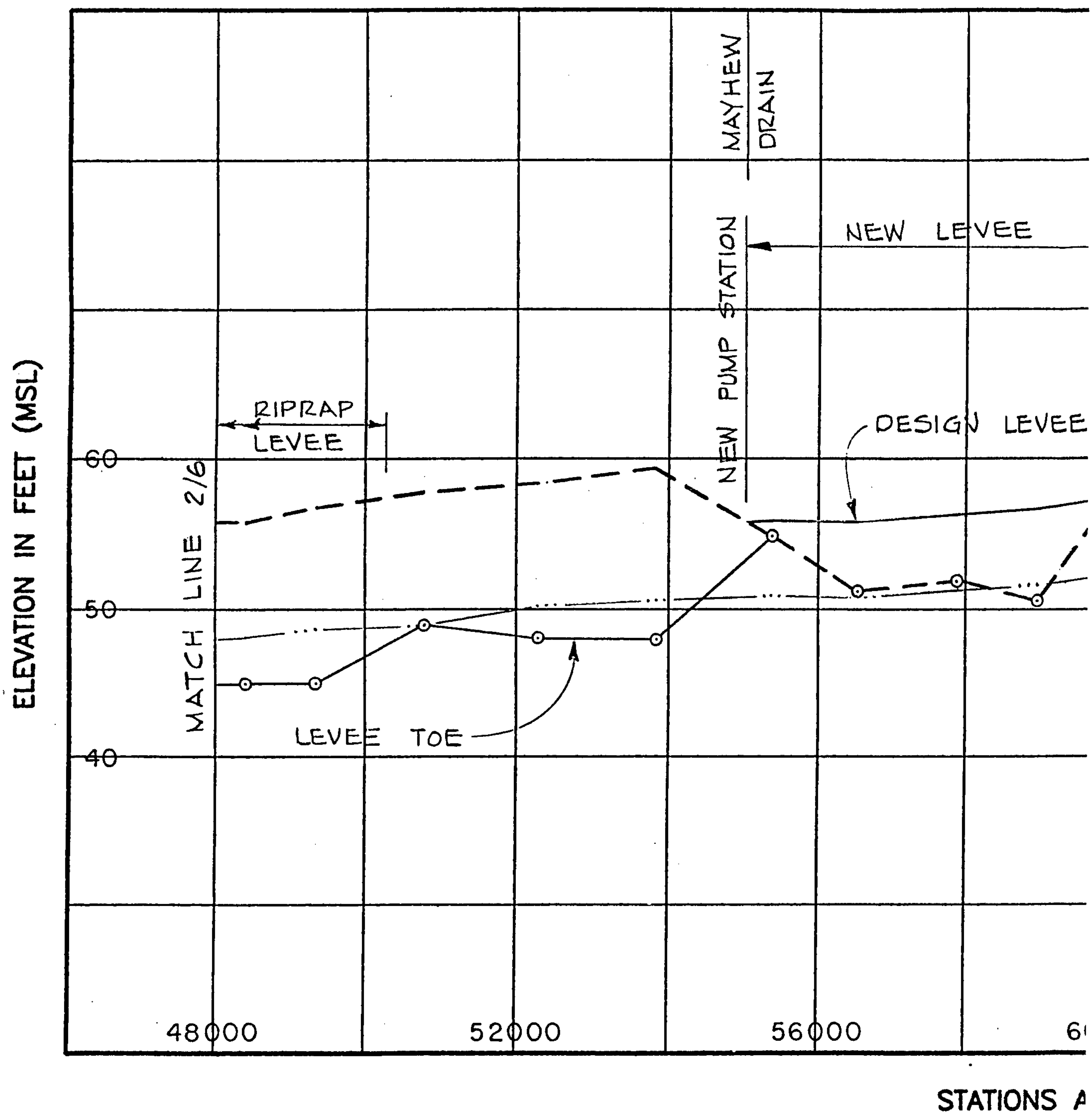
RLINE

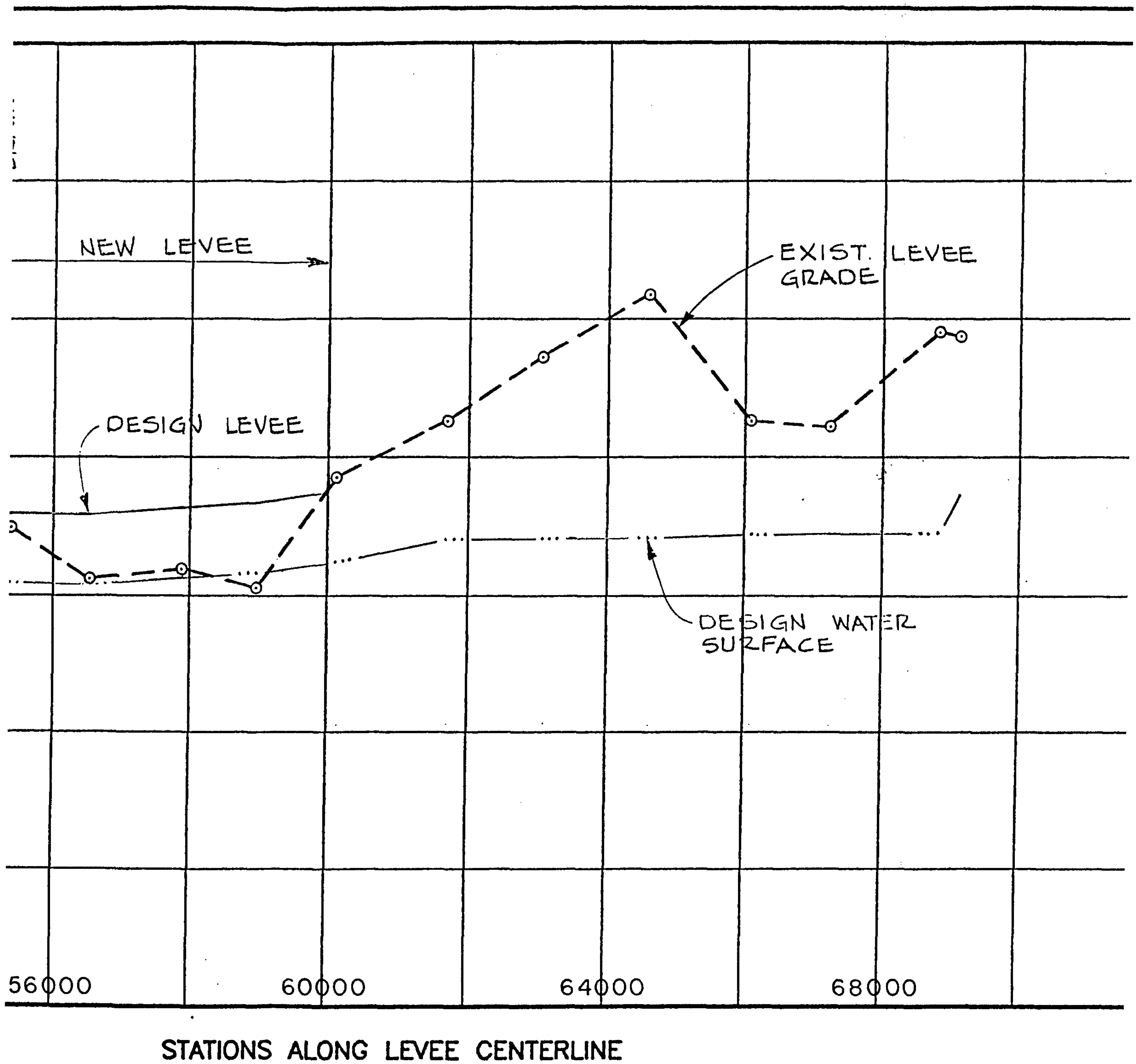
AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES
LOWER AMERICAN RIVER
SOUTH (LEFT) LEVEE PROFILE
130,000 CFS PLAN

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 2 2/6


ELEVATION IN FEET (MSL)
MAYHEW DRAIN
NEW PUMP STATION
NEW LEVEE
RIPRAP LEVEE
DESIGN LEVEE
MATCH LINE 2/6
LEVEE TOE
60
50
40
48000
52000
56000
6
STATIONS A



AMERICAN RIV

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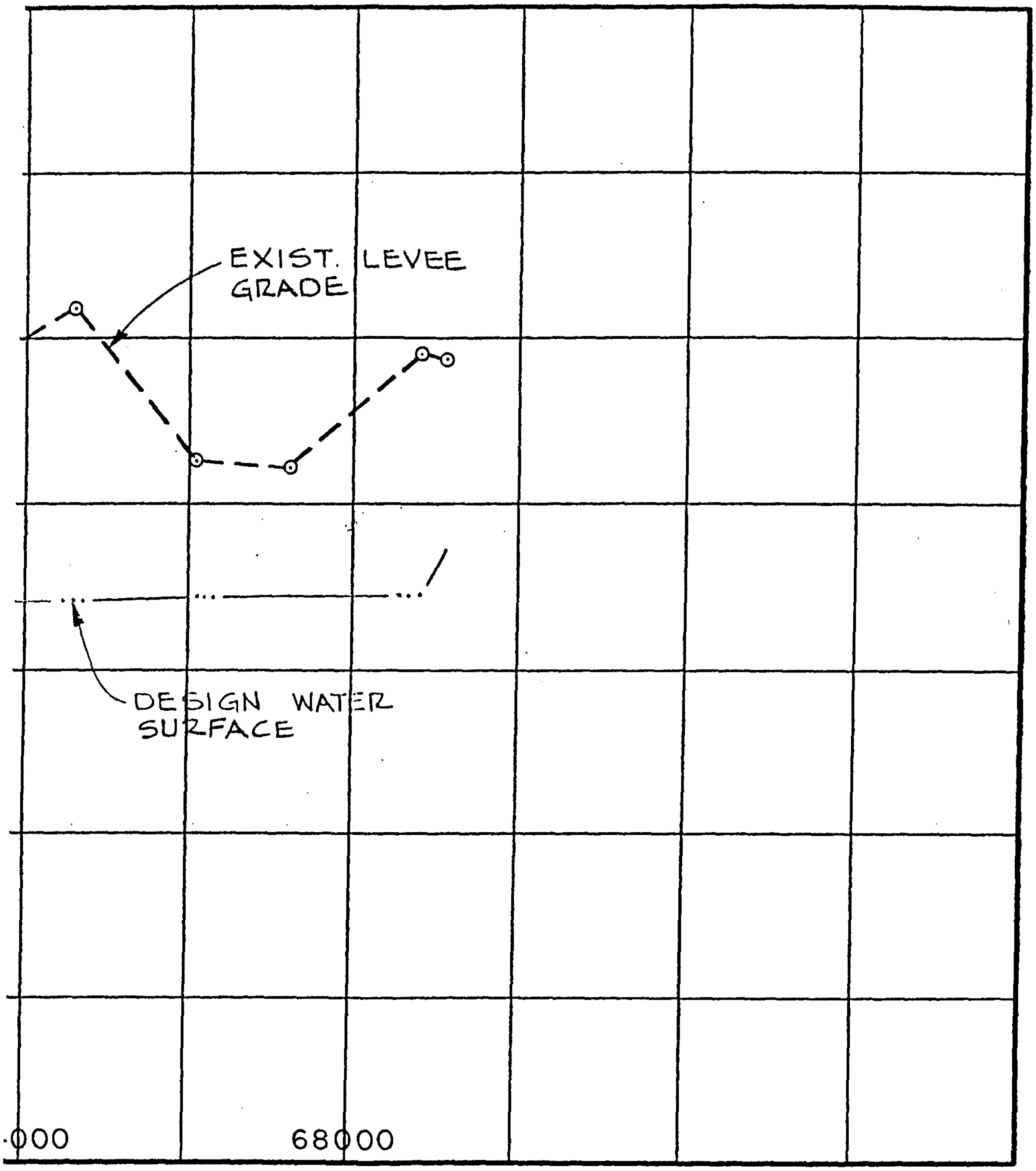
SOUTH (L
130,
SACRAMENTO

2



NE

AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

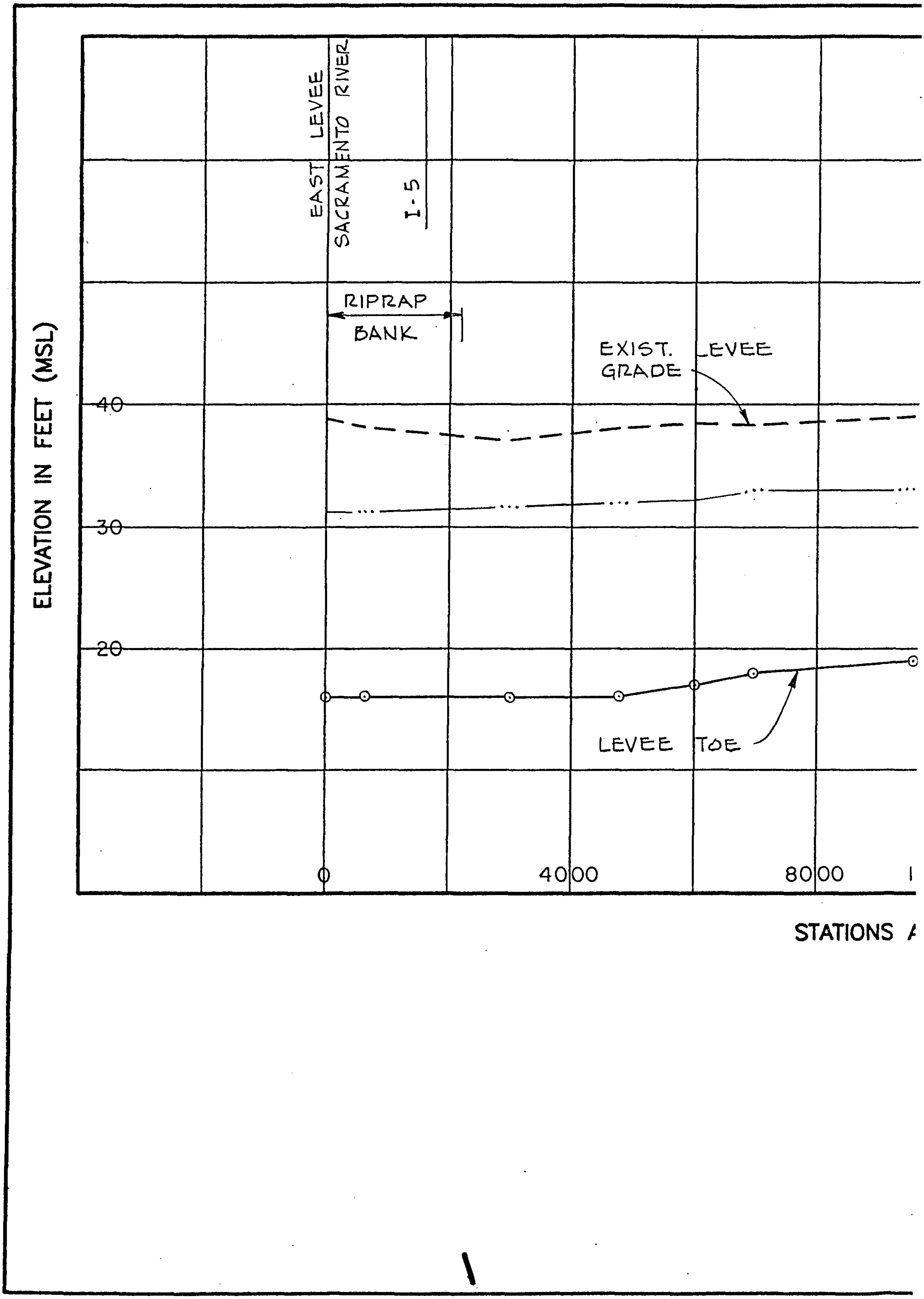
LEVEE CROWN AND
WATER SURFACE PROFILES

LOWER AMERICAN RIVER
SOUTH (LEFT) LEVEE PROFILE
130,000 CFS PLAN

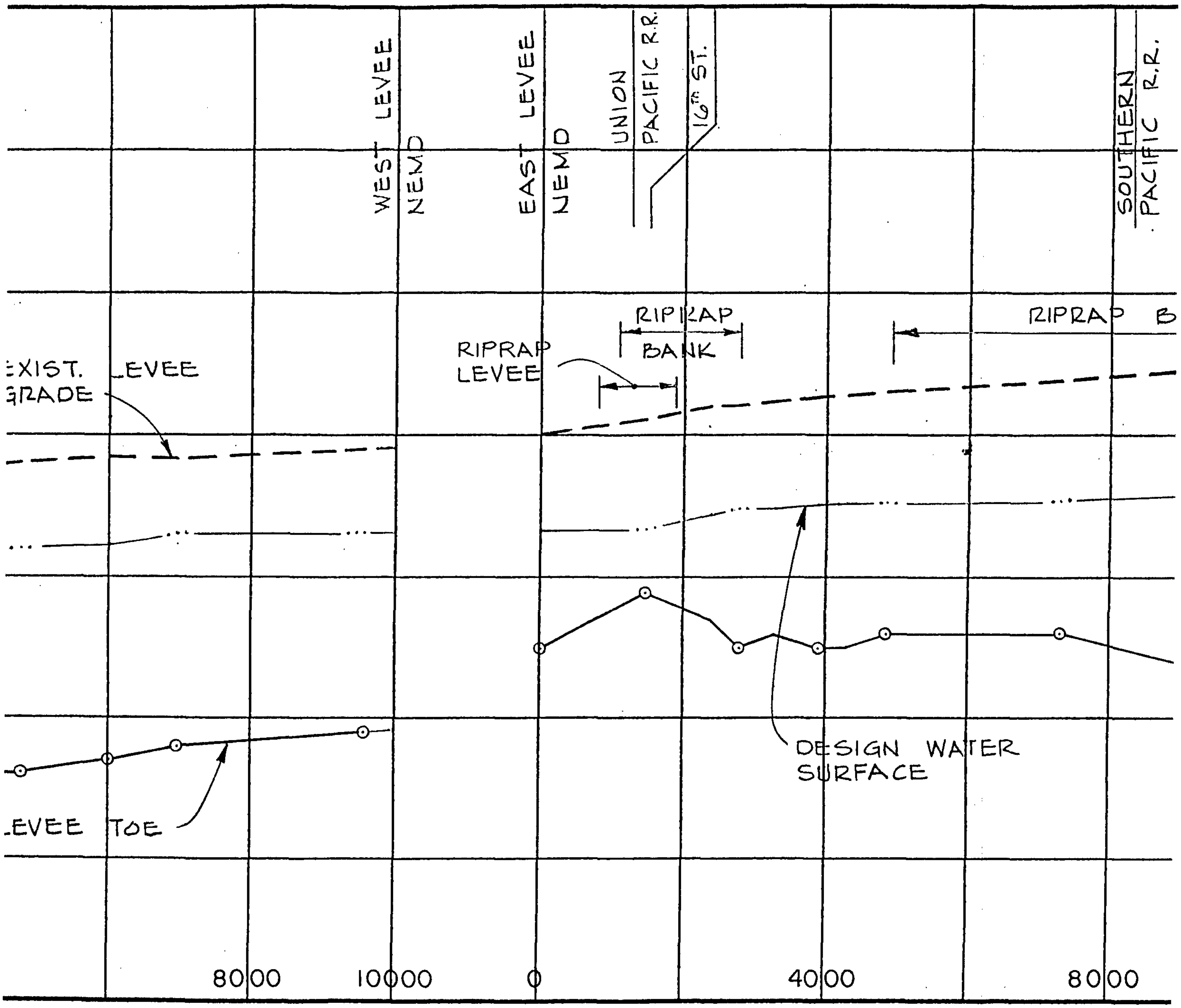
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 2 3/6

3



EAST LEVEE
SACRAMENTO RIVER
I-5
RIPRAP
BANK
EXIST. LEVEE
GRADE
LEVEE TOE
40
30
20
0
40|00
80|00
ELEVATION IN FEET (MSL)
STATIONS



STATIONS ALONG LEVEE CENTERLINE

AMERICA

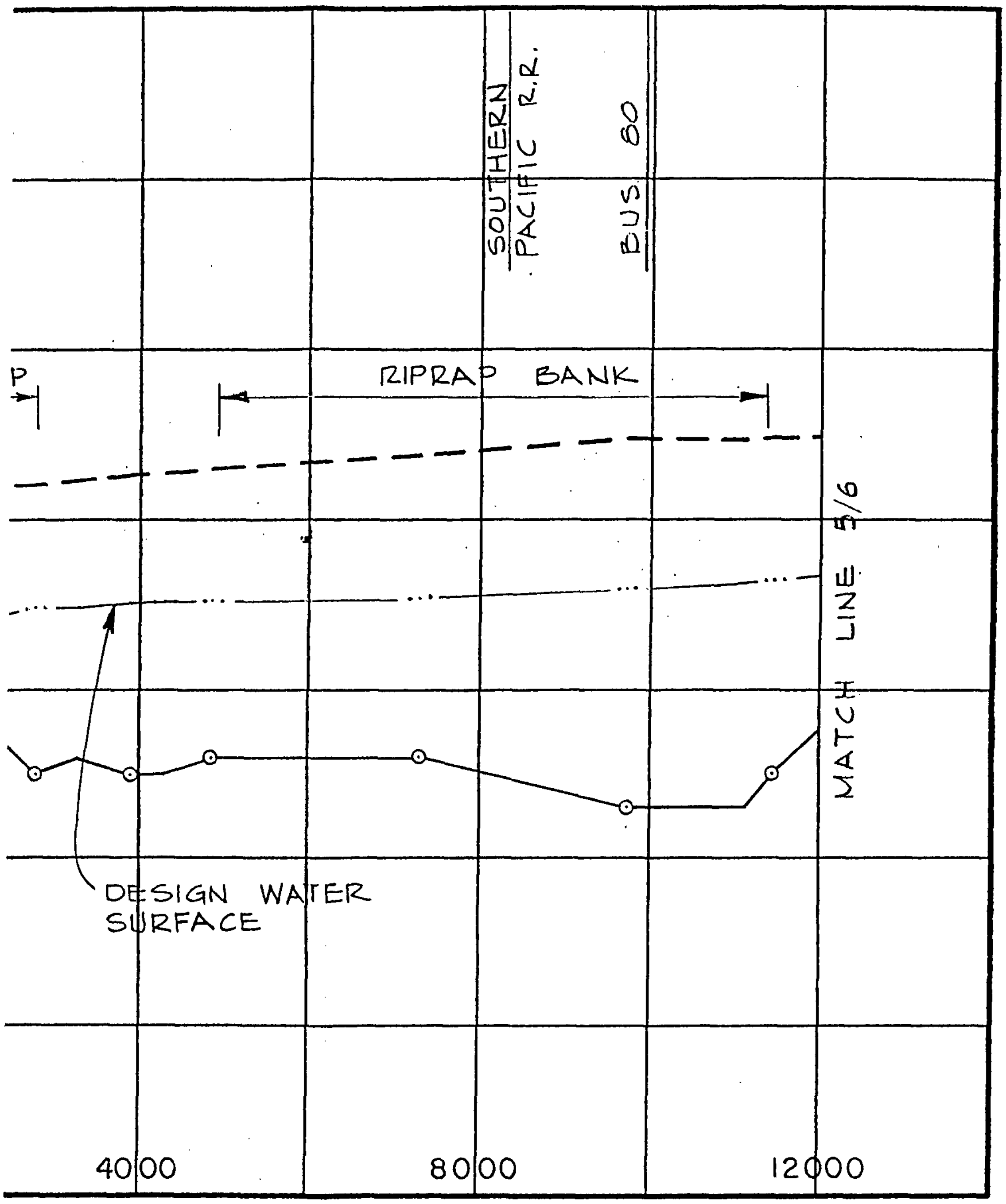
W

LOWE

NORTH

13

SACRAME



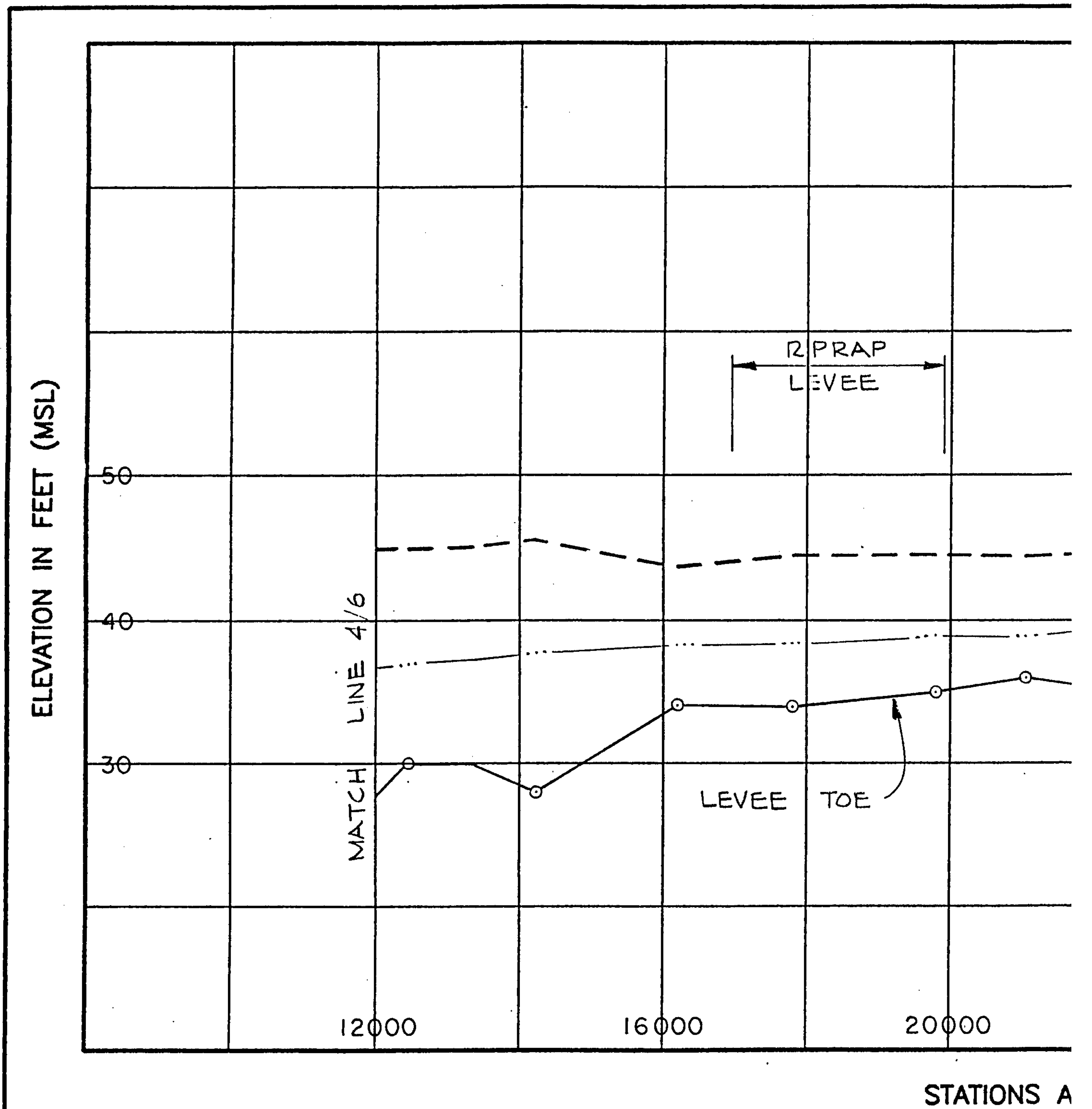
AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES
LOWER AMERICAN RIVER
NORTH (RIGHT) LEVEE PROFILE
130,000 CFS PLAN

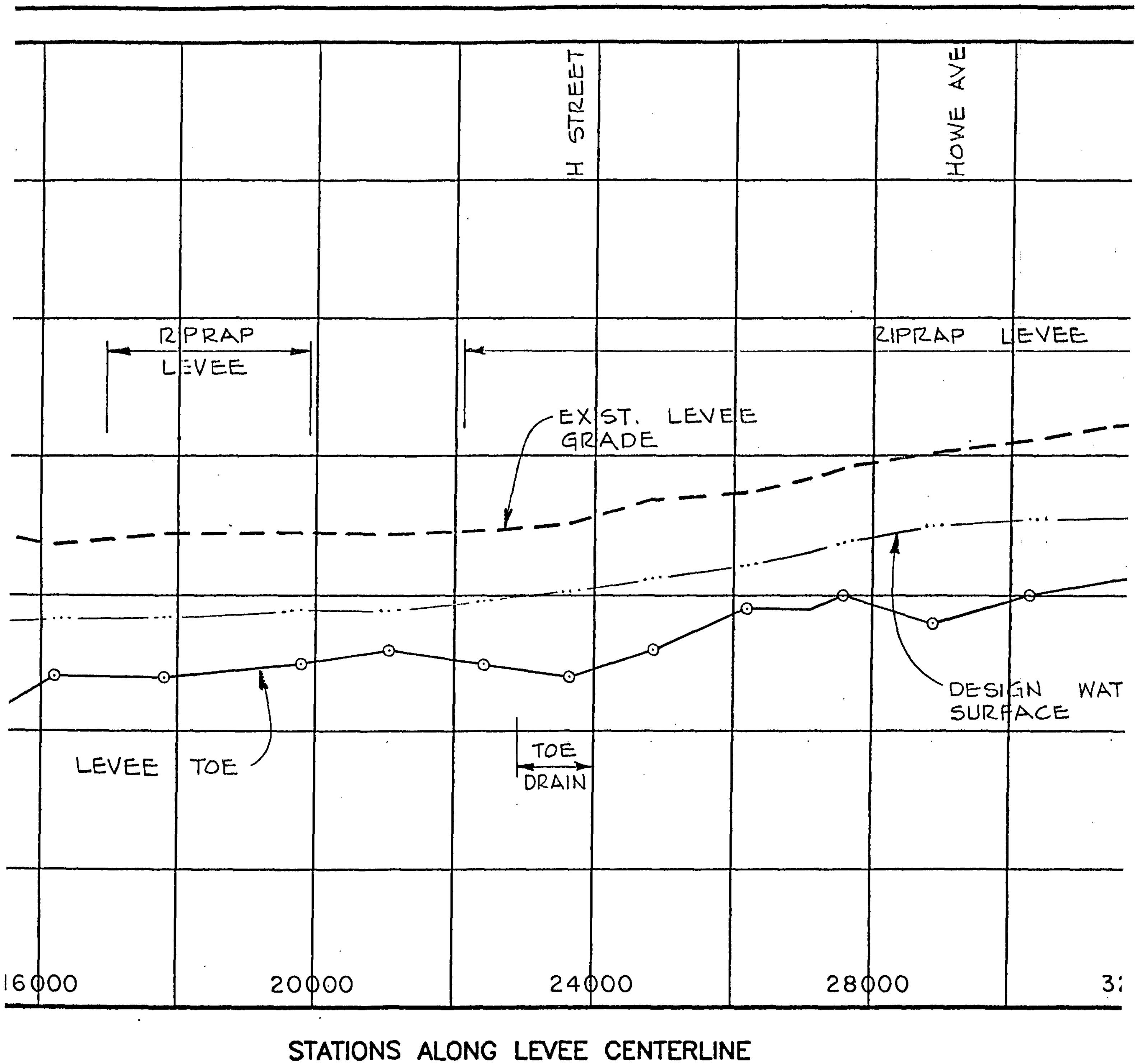
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

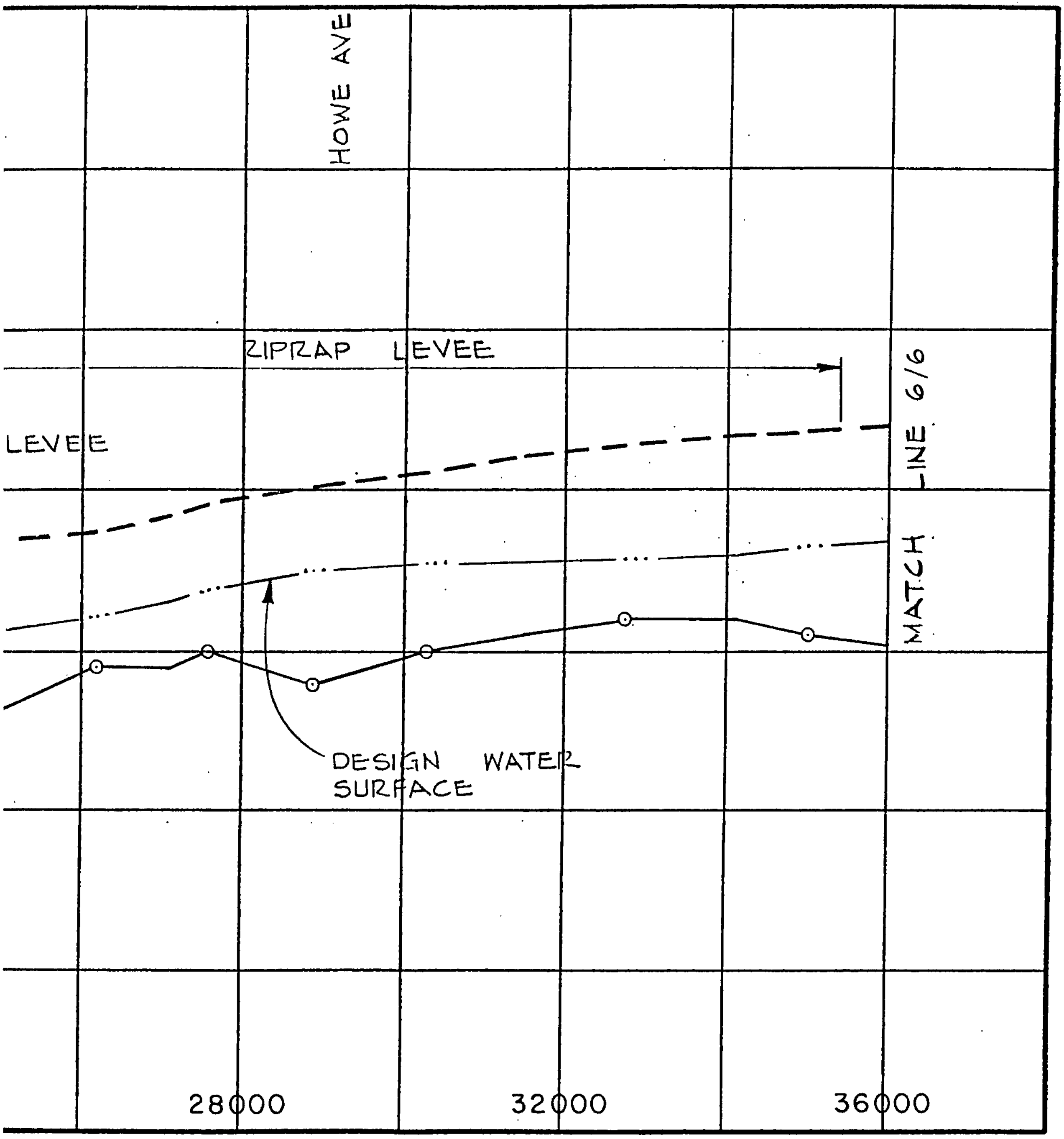
PLATE 2 4/6

3



ELEVATION IN FEET (MSL)
50
40
30
MATCH LINE 4/6
RIPRAP
LEVEE
LEVEE TOE
12000
16000
20000
STATIONS A


H STREET
HOWE AVE
RIPRAP LEVEE
RIPRAP LEVEE
EXIST. LEVEE GRADE
DESIGN WAT
SURFACE
LEVEE TOE
TOE DRAIN
16000
20000
24000
28000
STATIONS ALONG LEVEE CENTERLINE



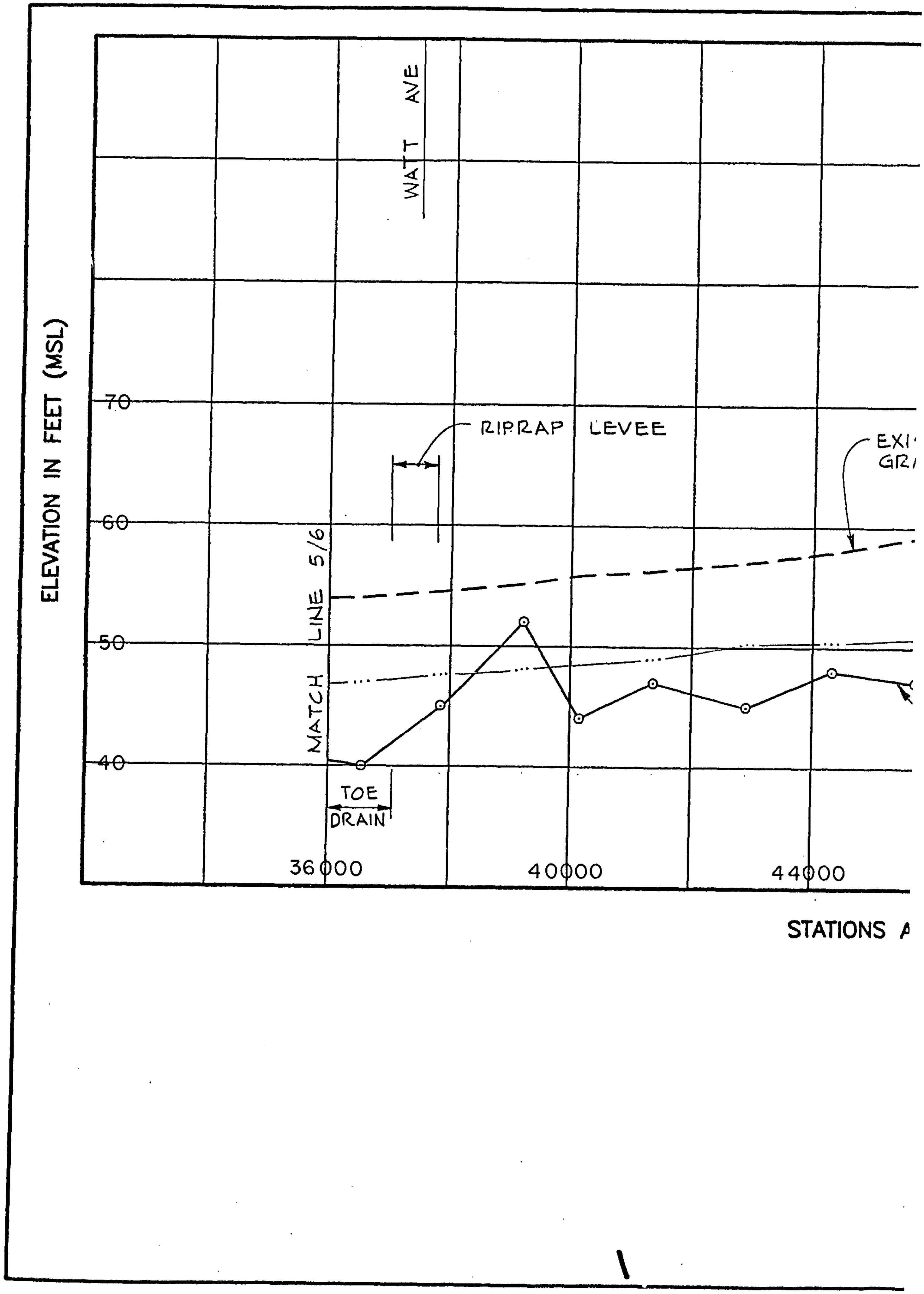
ERLINE

AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

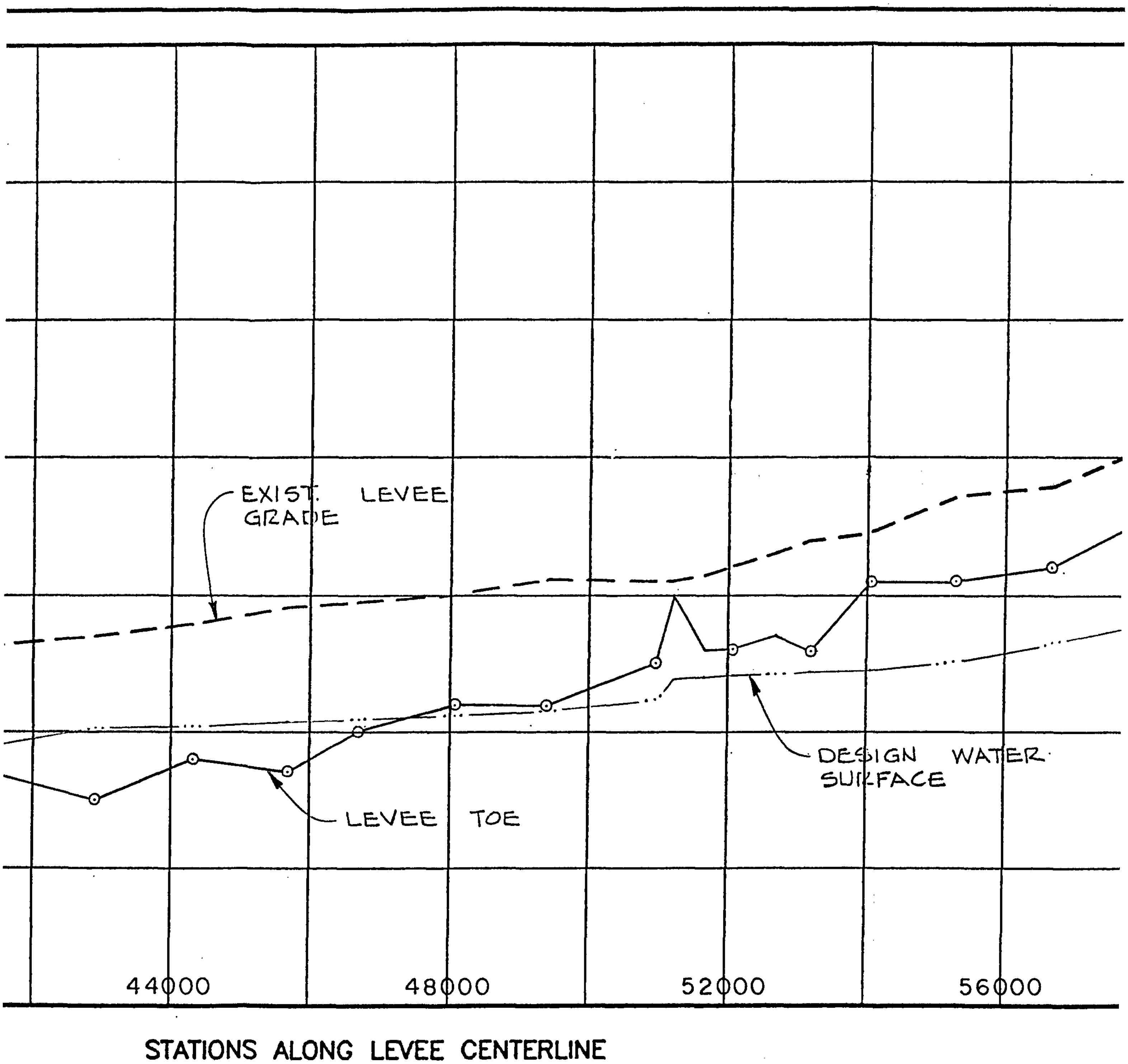
LEVEE CROWN AND
WATER SURFACE PROFILES
LOWER AMERICAN RIVER
NORTH (RIGHT) LEVEE PROFILE
130,000 CFS PLAN
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

3

PLATE 2 5/6



ELEVATION IN FEET (MSL)
70
60
50
40
WATT AVE
RIPRAP LEVEE
EXI
GR
MATCH LINE 5/6
TOE DRAIN
36000
40000
44000
STATIONS



AMERICAN RIVE

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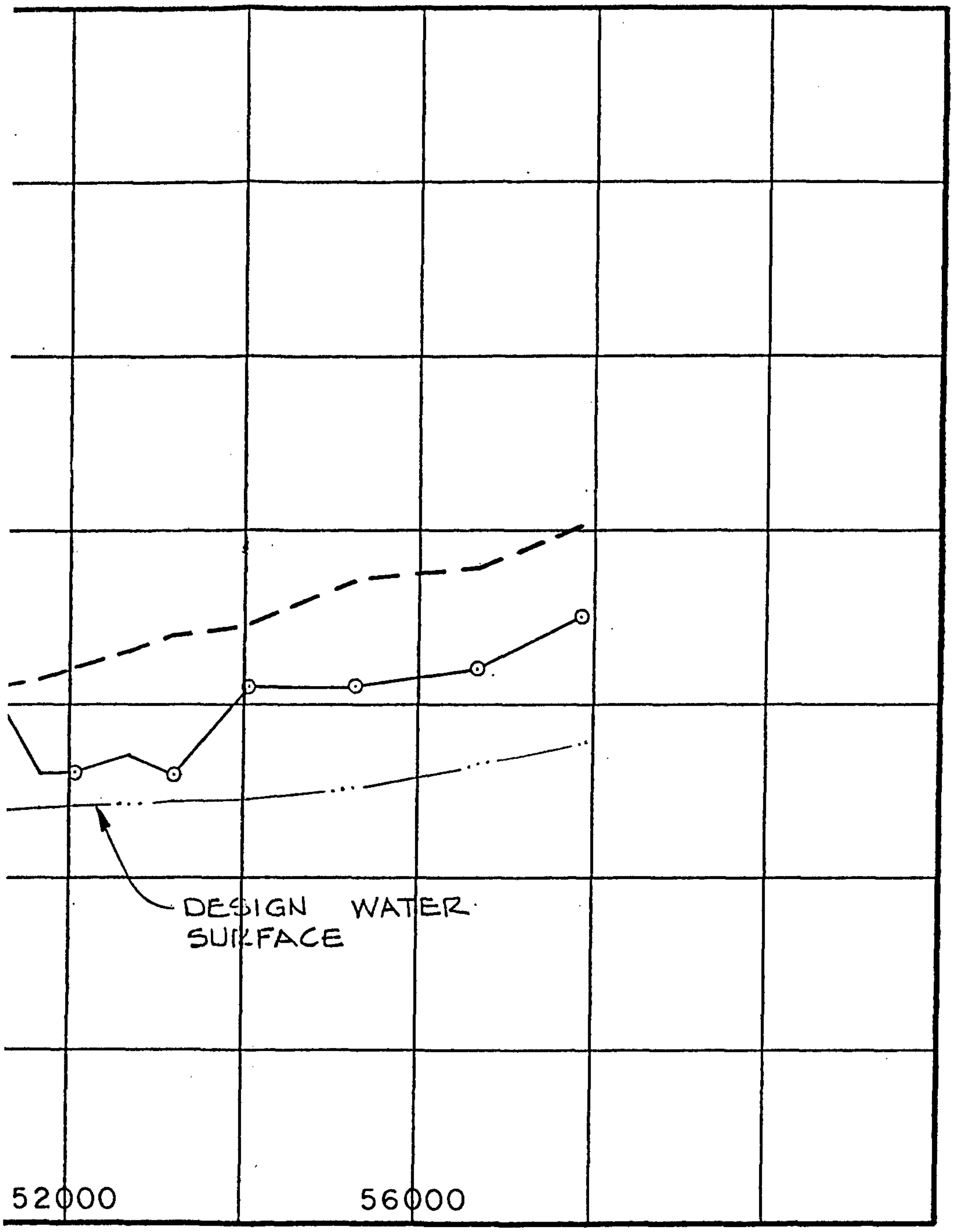
WATER

LOWER

NORTH (RI

130,0

SACRAMENTO DI



AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES

LOWER AMERICAN RIVER

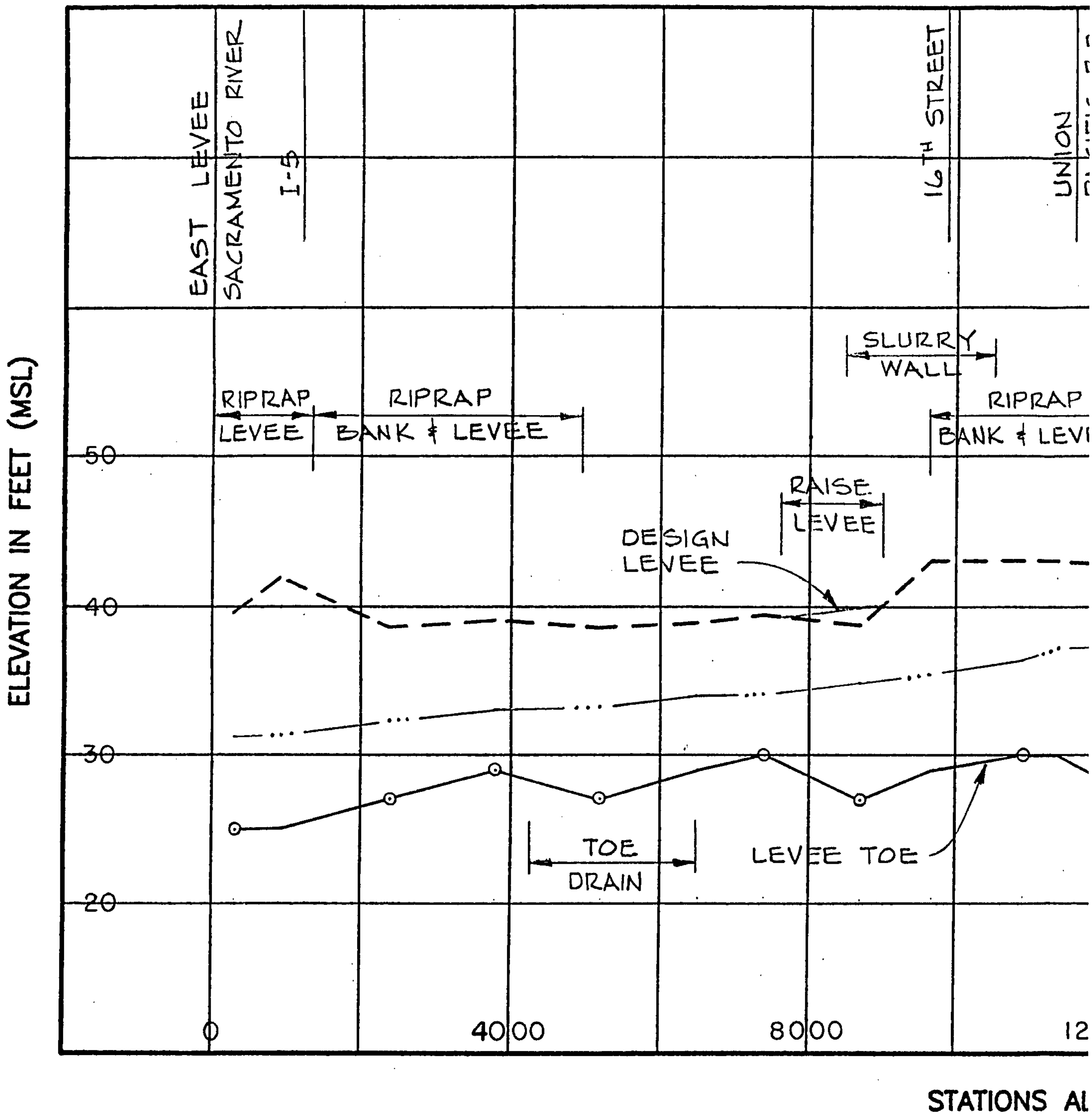
NORTH (RIGHT) LEVEE PROFILE

130,000 CFS PLAN

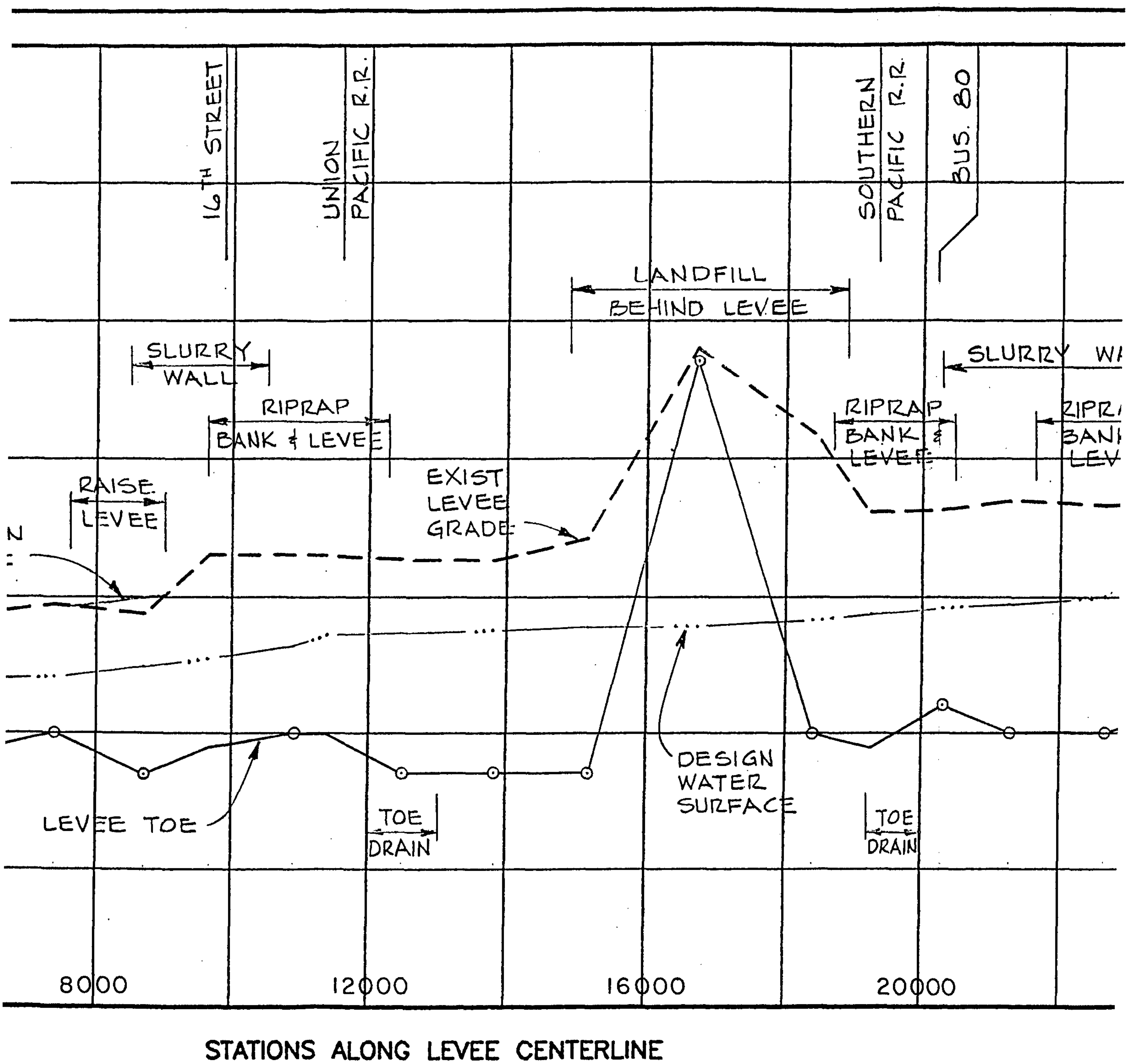
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

3

PLATE 2 6/6


EAST LEVEE
SACRAMENTO RIVER
I-5
16TH STREET
UNION
SLURRY WALL
RIPRAP LEVEE
RIPRAP BANK & LEVEE
RIPRAP BANK & LEV
RAISE LEVEE
DESIGN LEVEE
TOE DRAIN
LEVEE TOE
ELEVATION IN FEET (MSL)
50
40
30
20
0
4000
8000
12
STATIONS AL

\



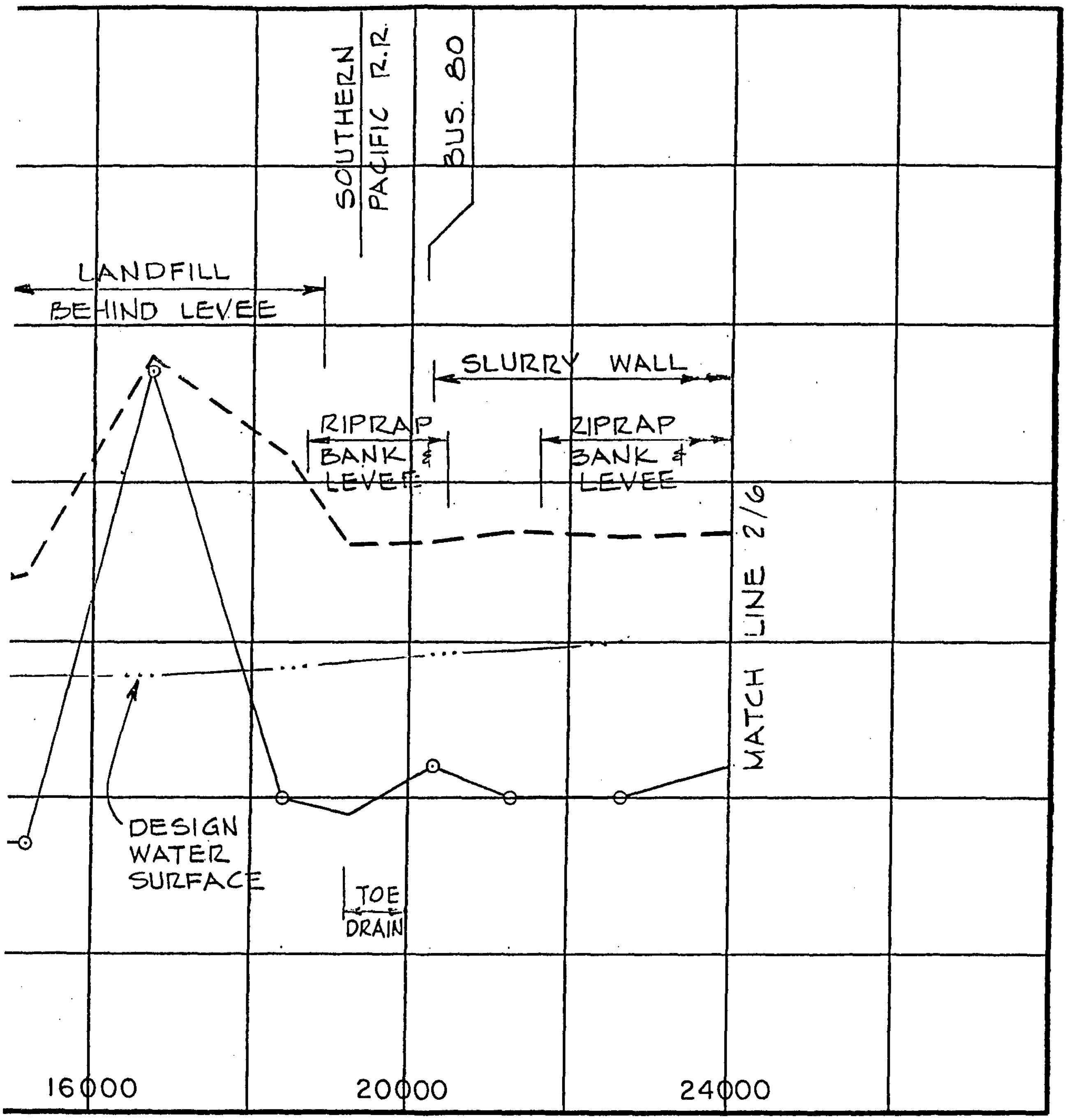
AMERIC

W

LOW

SOUTH

SACRAME



TERLINE

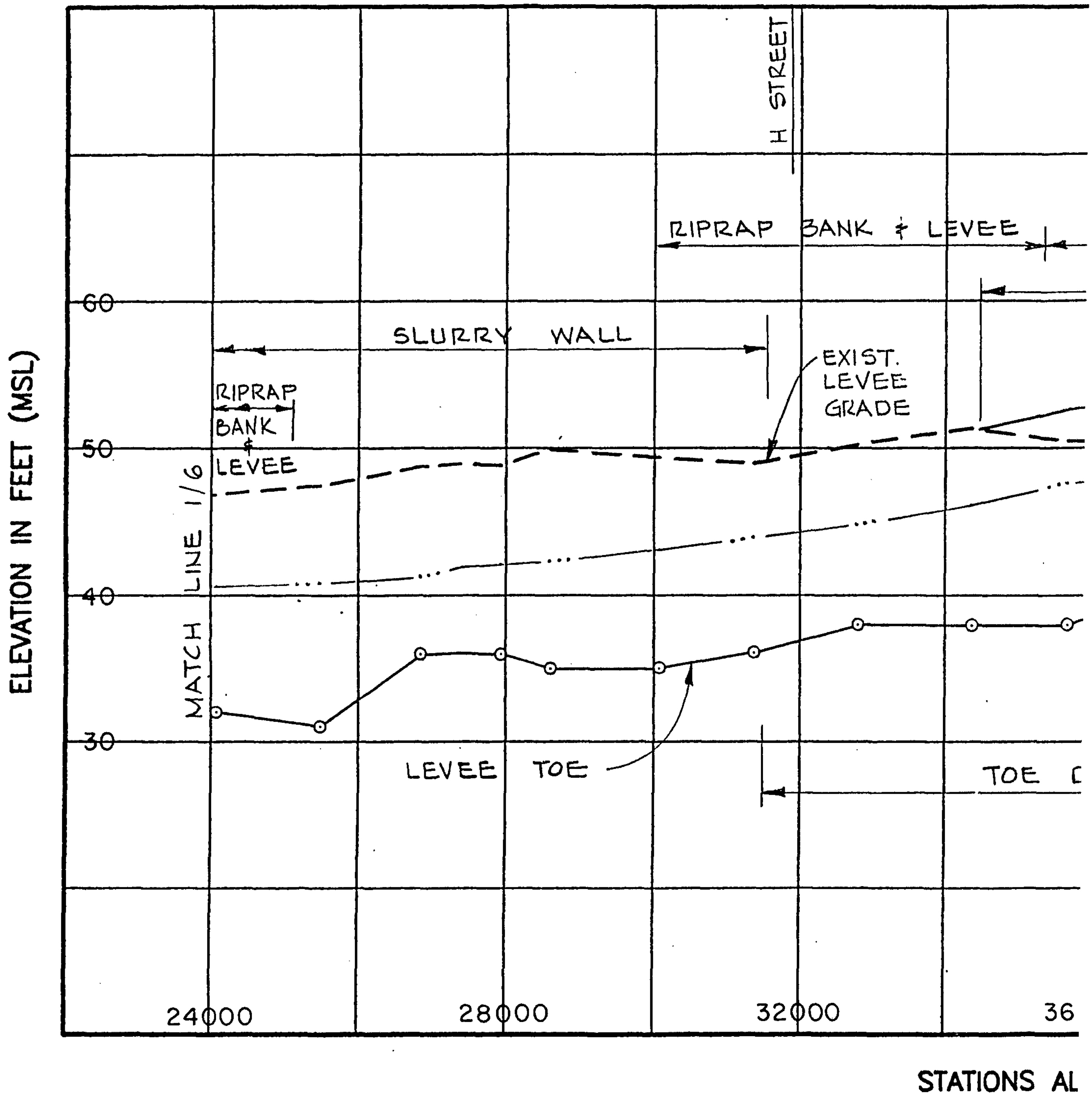
AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES

LOWER AMERICAN RIVER
SOUTH (LEFT) LEVEE PROFILE
180,000 CFS PLAN

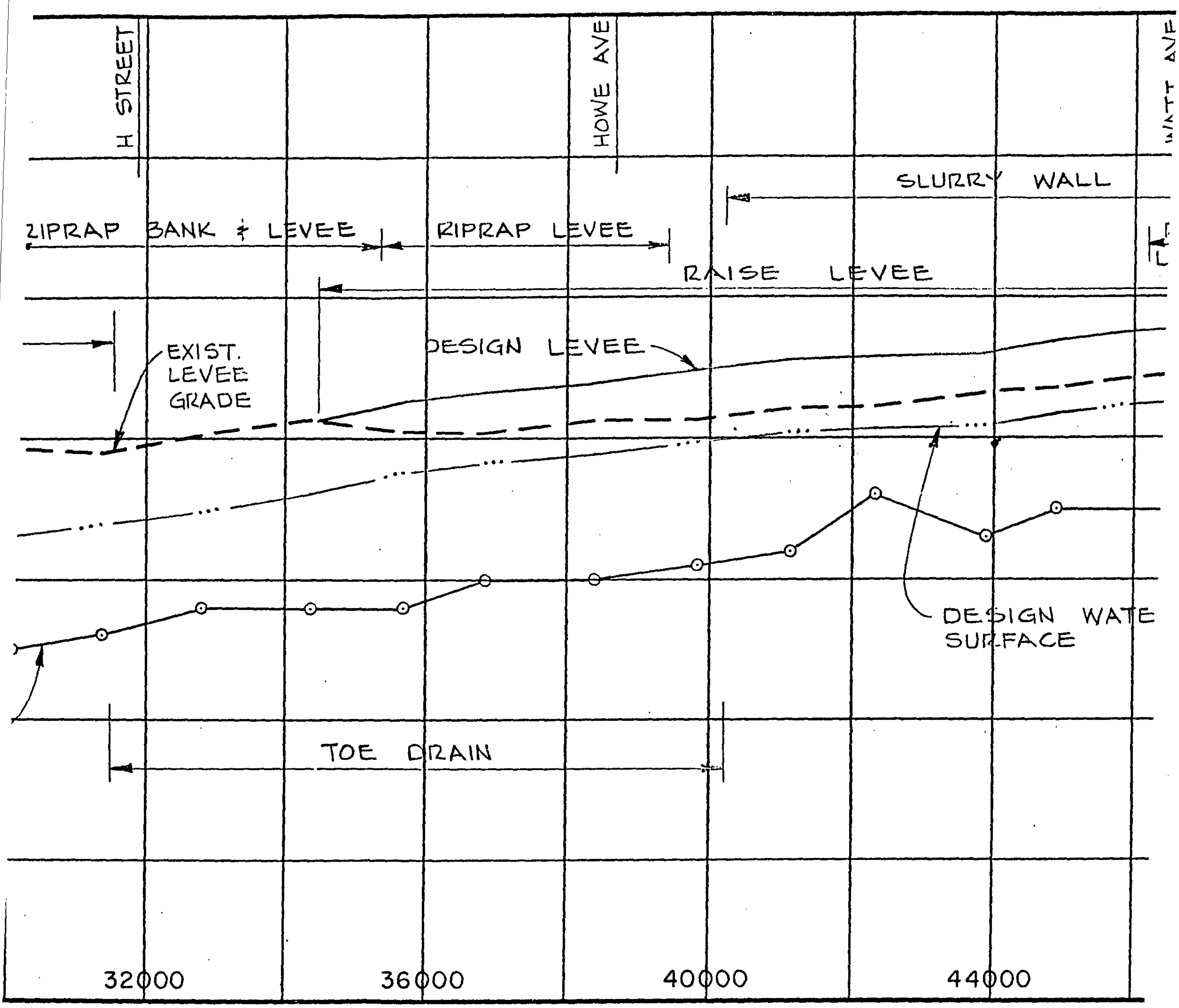
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 3 1/6

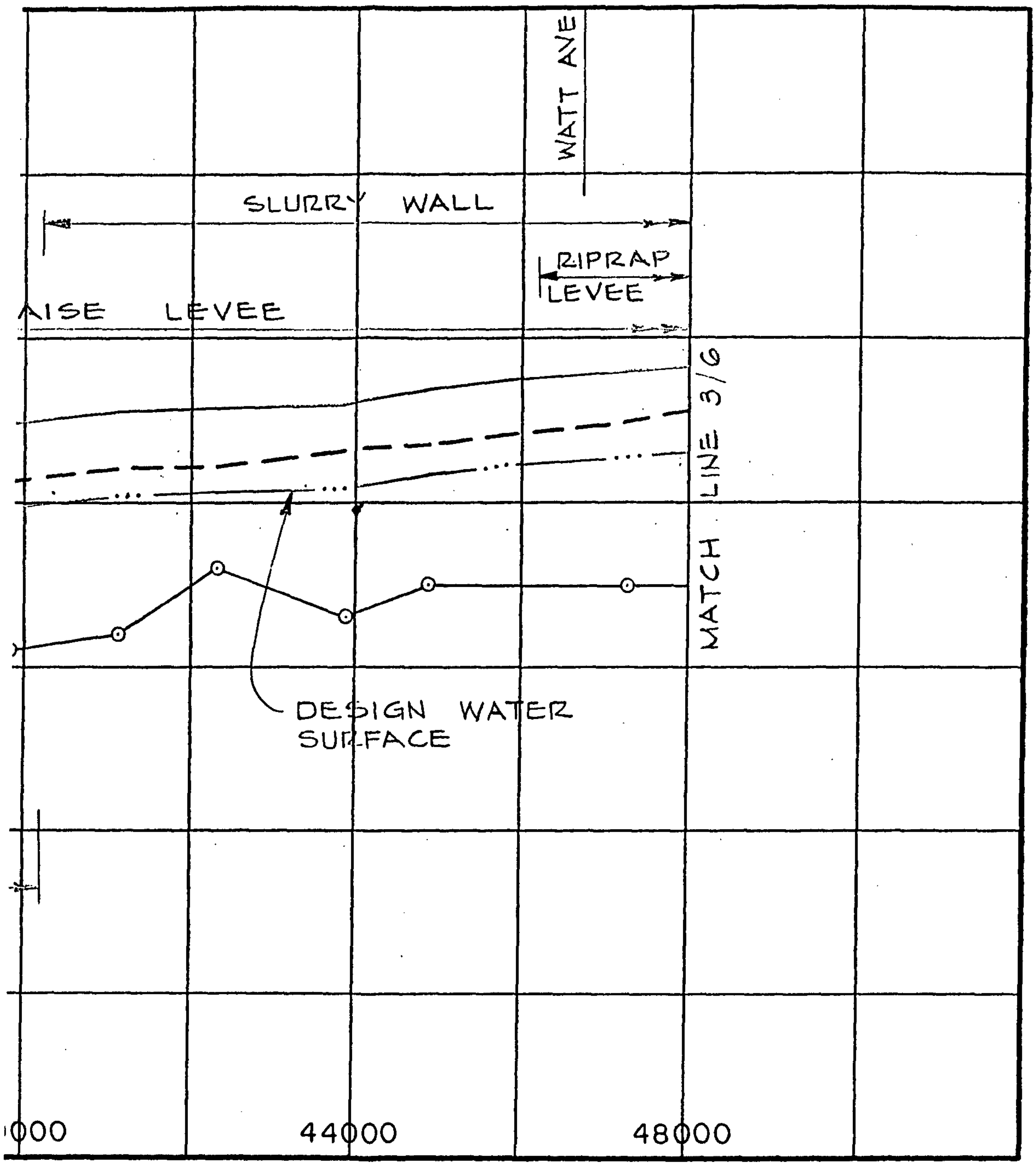


H STREET
RIPRAP BANK & LEVEE
SLURRY WALL
RIPRAP BANK & LEVEE
EXIST. LEVEE GRADE
MATCH LINE 1/6
LEVEE TOE
TOE
60
50
40
30
24000
28000
32000
36
ELEVATION IN FEET (MSL)
STATIONS AL

1



STATIONS ALONG LEVEE CENTERLINE



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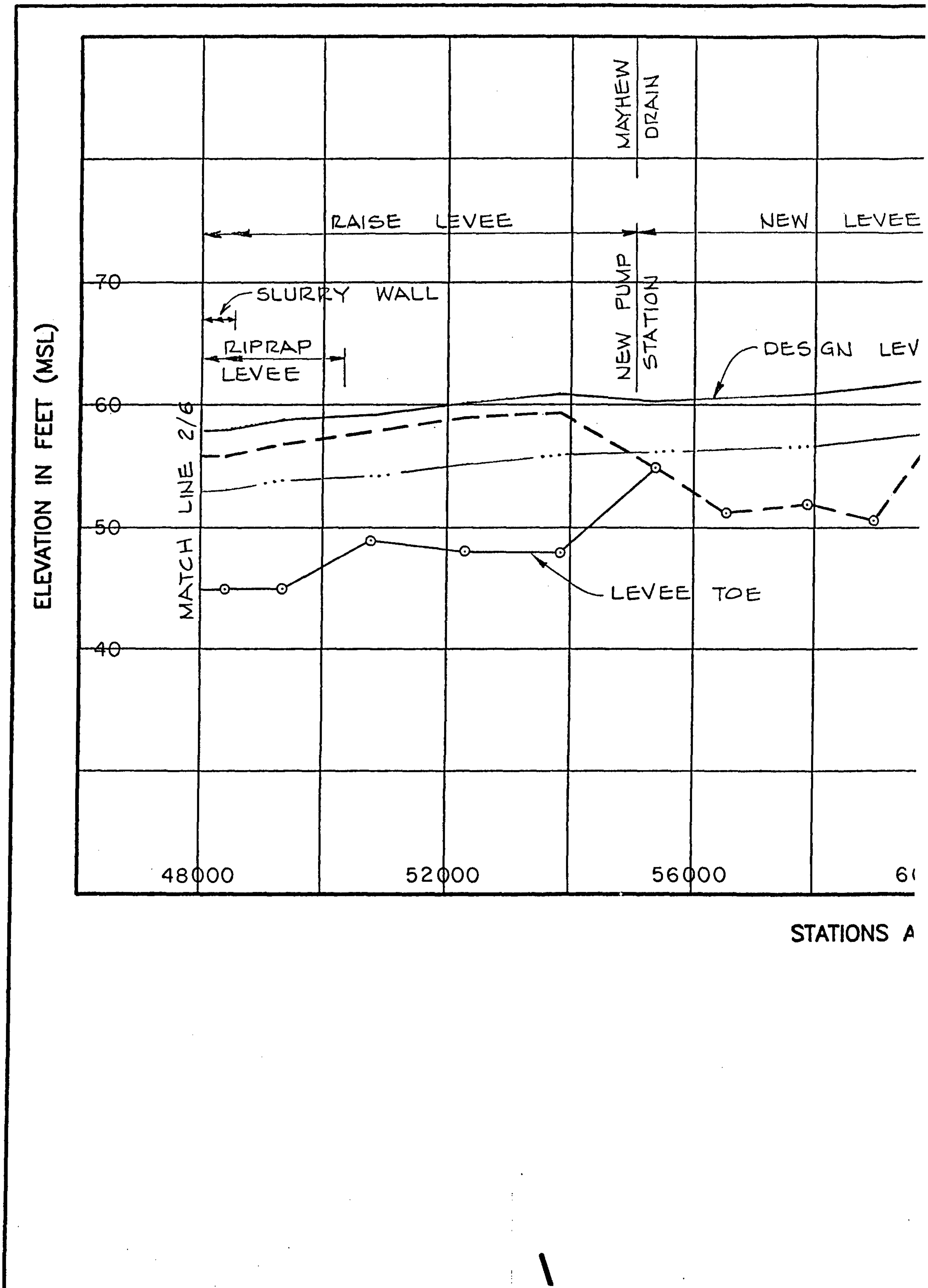
AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES
LOWER AMERICAN RIVER
SOUTH (LEFT) LEVEE PROFILE
180,000 CFS PLAN

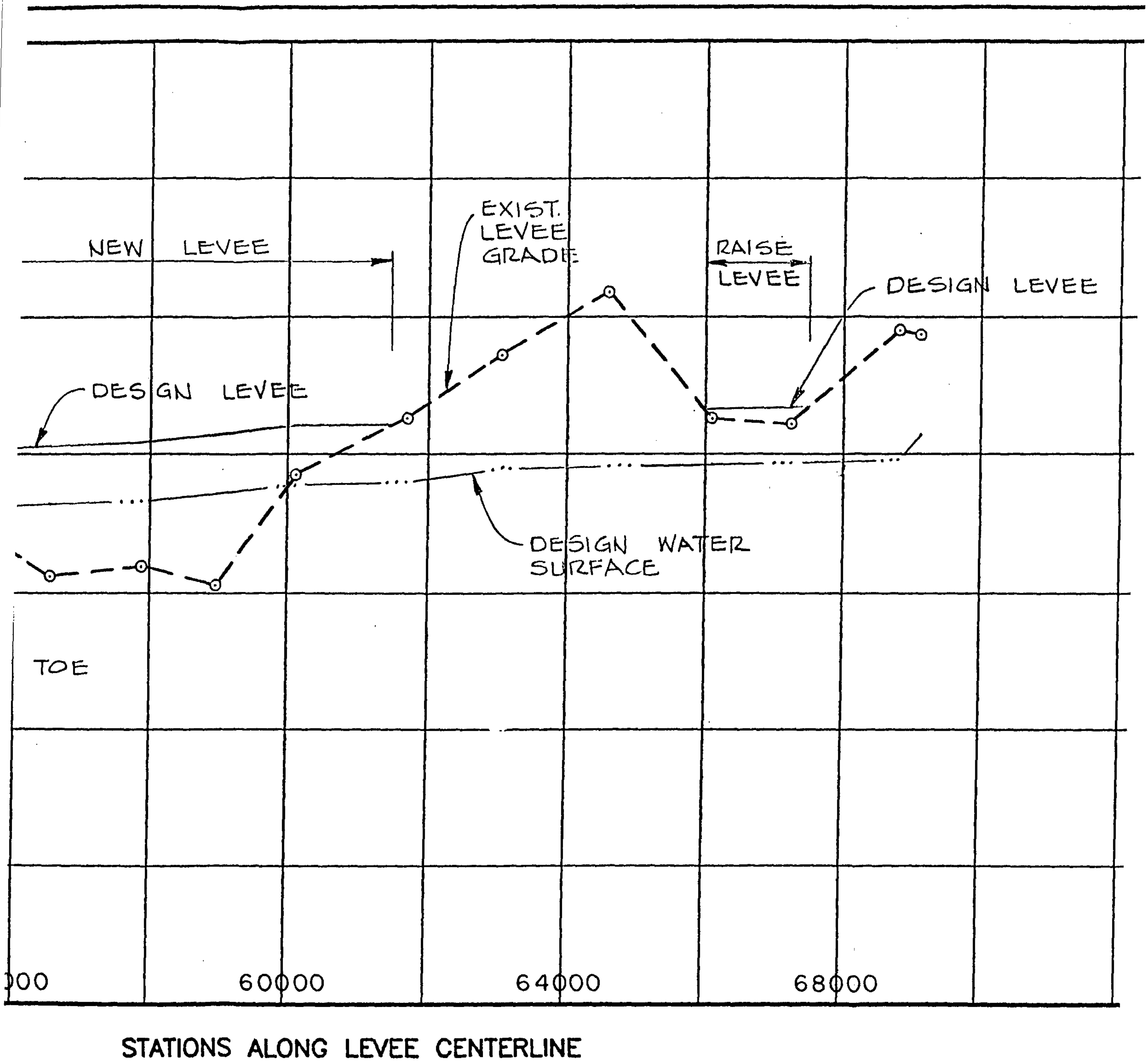
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

3

PLATE 3 2/6



MAYHEW
DRAIN
RAISE LEVEE
NEW LEVEE
70
SLURRY WALL
NEW PUMP
STATION
RIPRAP
LEVEE
DESIGN LEV
60
MATCH LINE 2/6
50
LEVEE TOE
40
48000
52000
56000
6(
ELEVATION IN FEET (MSL)
STATIONS A



AMERICAN RIVER

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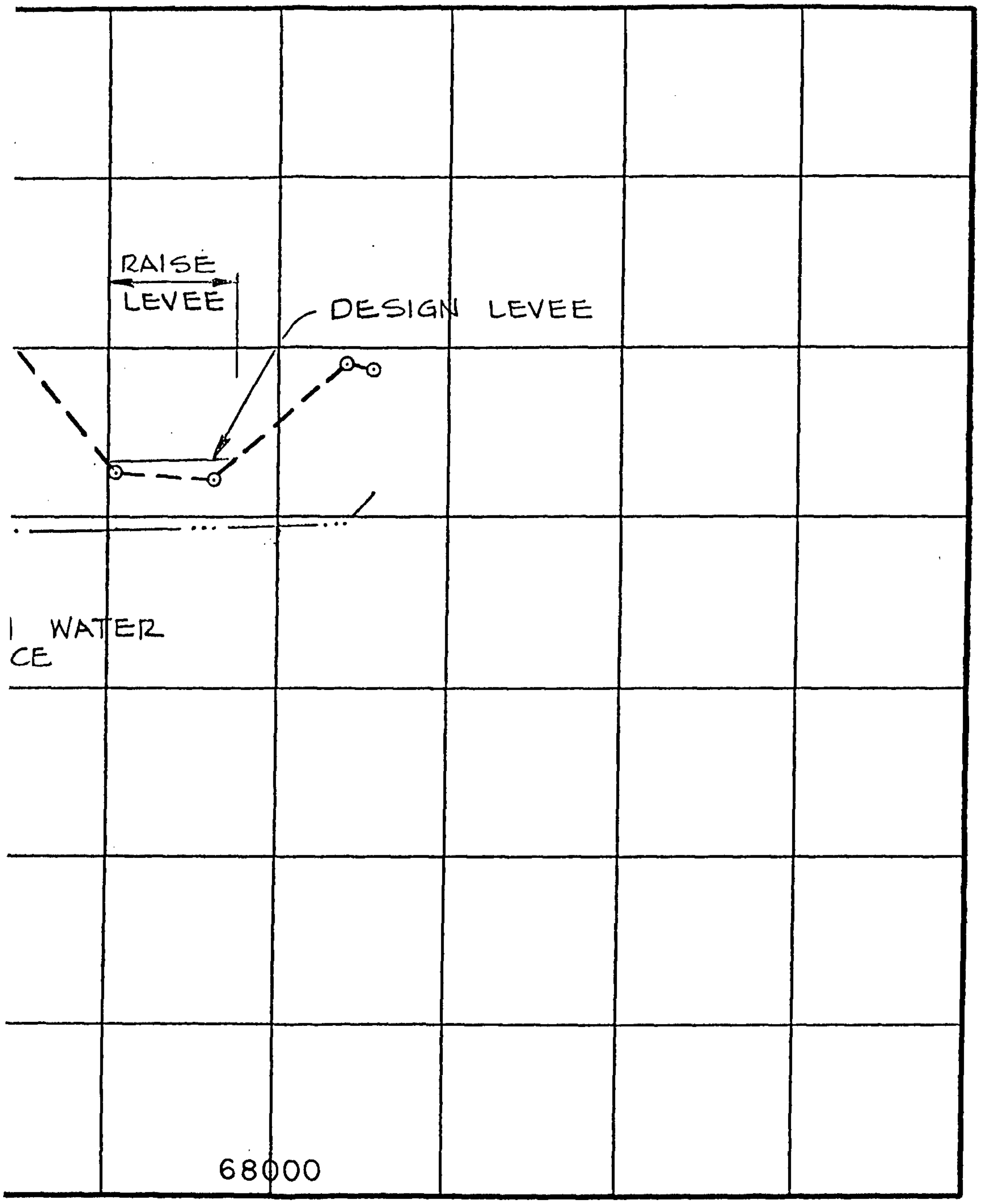
WATER S
LOWER A
SOUTH (LEI
180,00
SACRAMENTO DIST



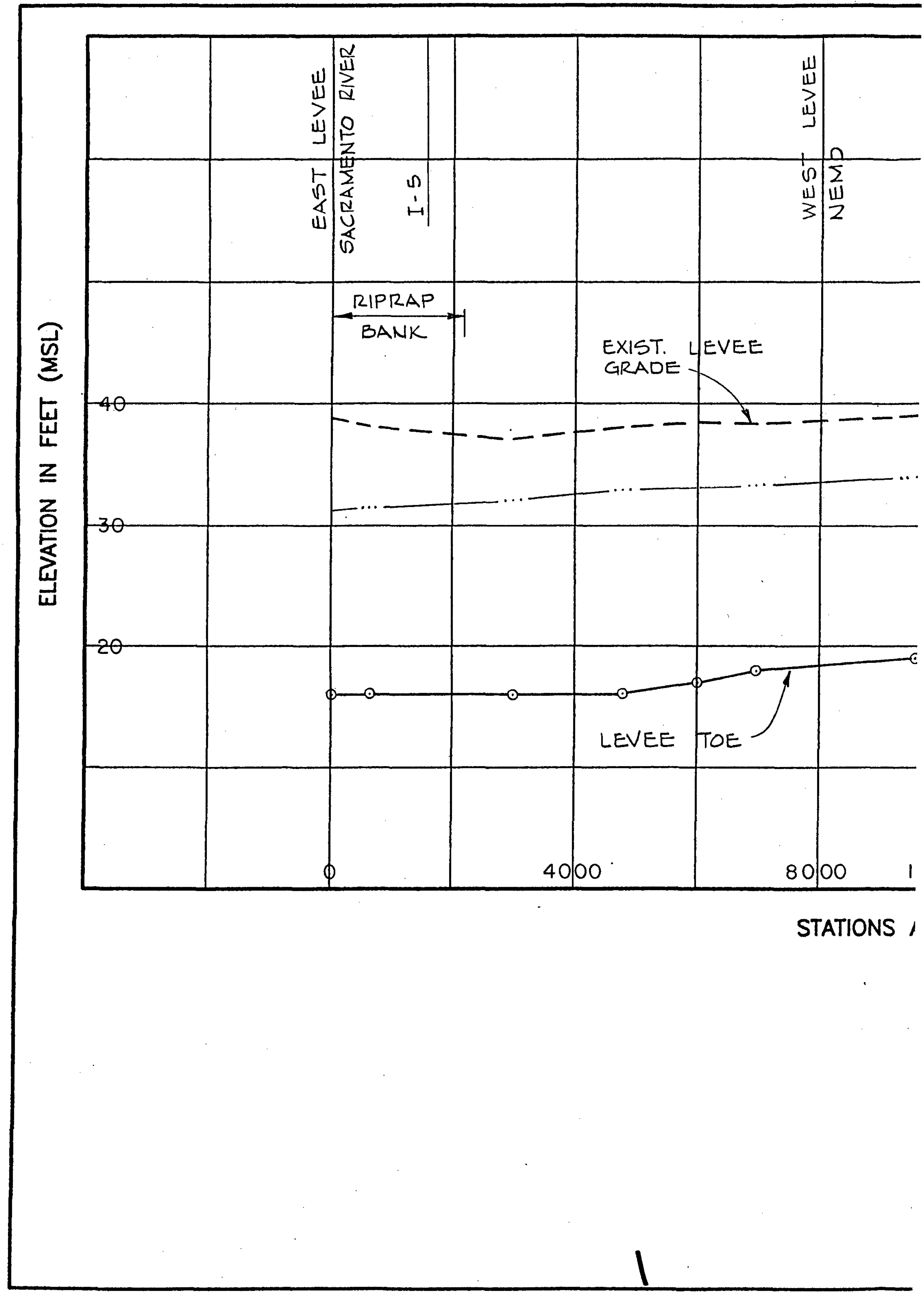
AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES
LOWER AMERICAN RIVER
SOUTH (LEFT) LEVEE PROFILE
180,000 CFS PLAN

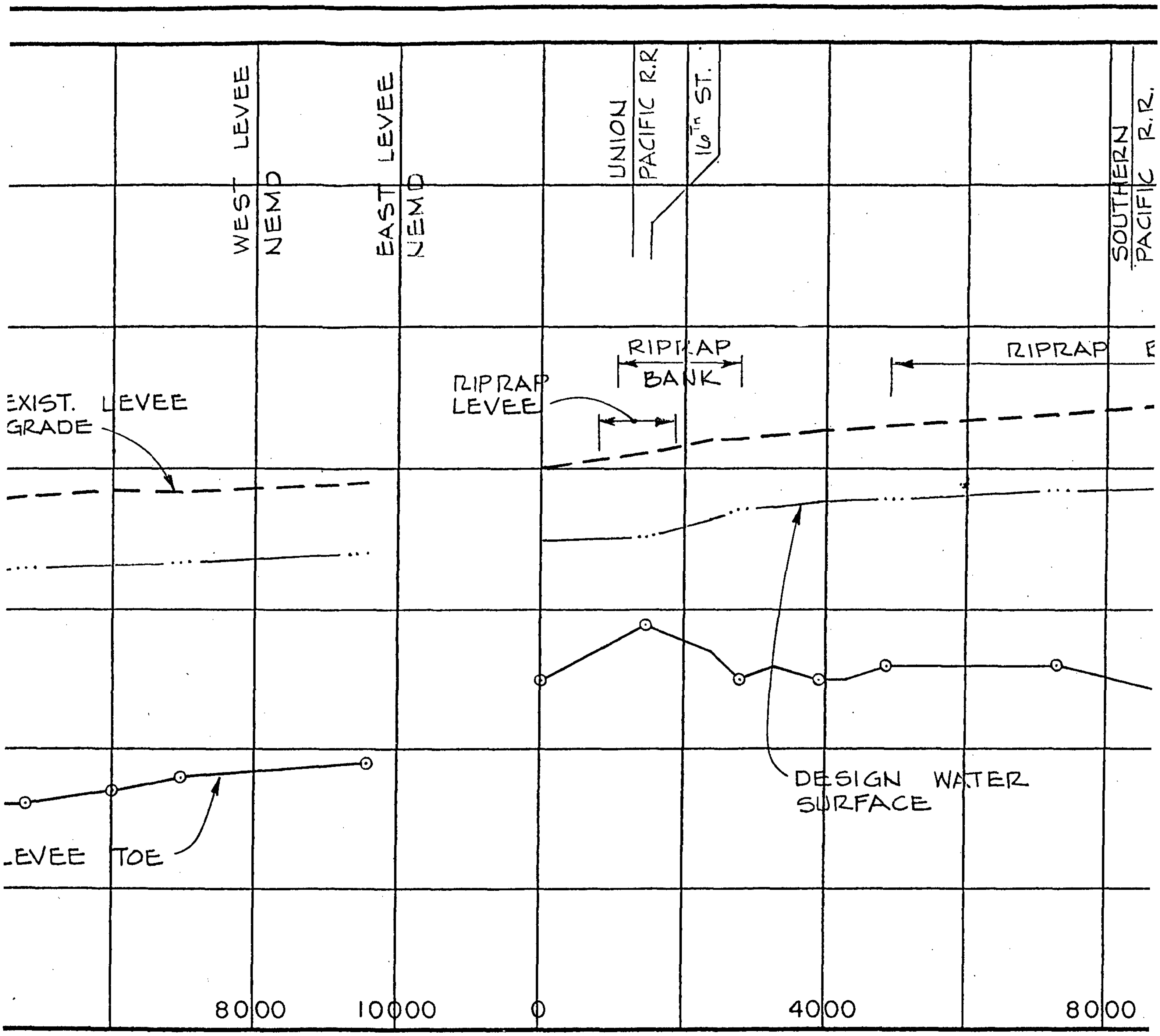
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 3 3/6

3



ELEVATION IN FEET (MSL)
EAST LEVEE
SACRAMENTO RIVER
I-5
WEST LEVEE
NEMD
RIPRAP
BANK
EXIST. LEVEE
GRADE
LEVEE TOE
40
30
20
0
4000
8000
STATIONS

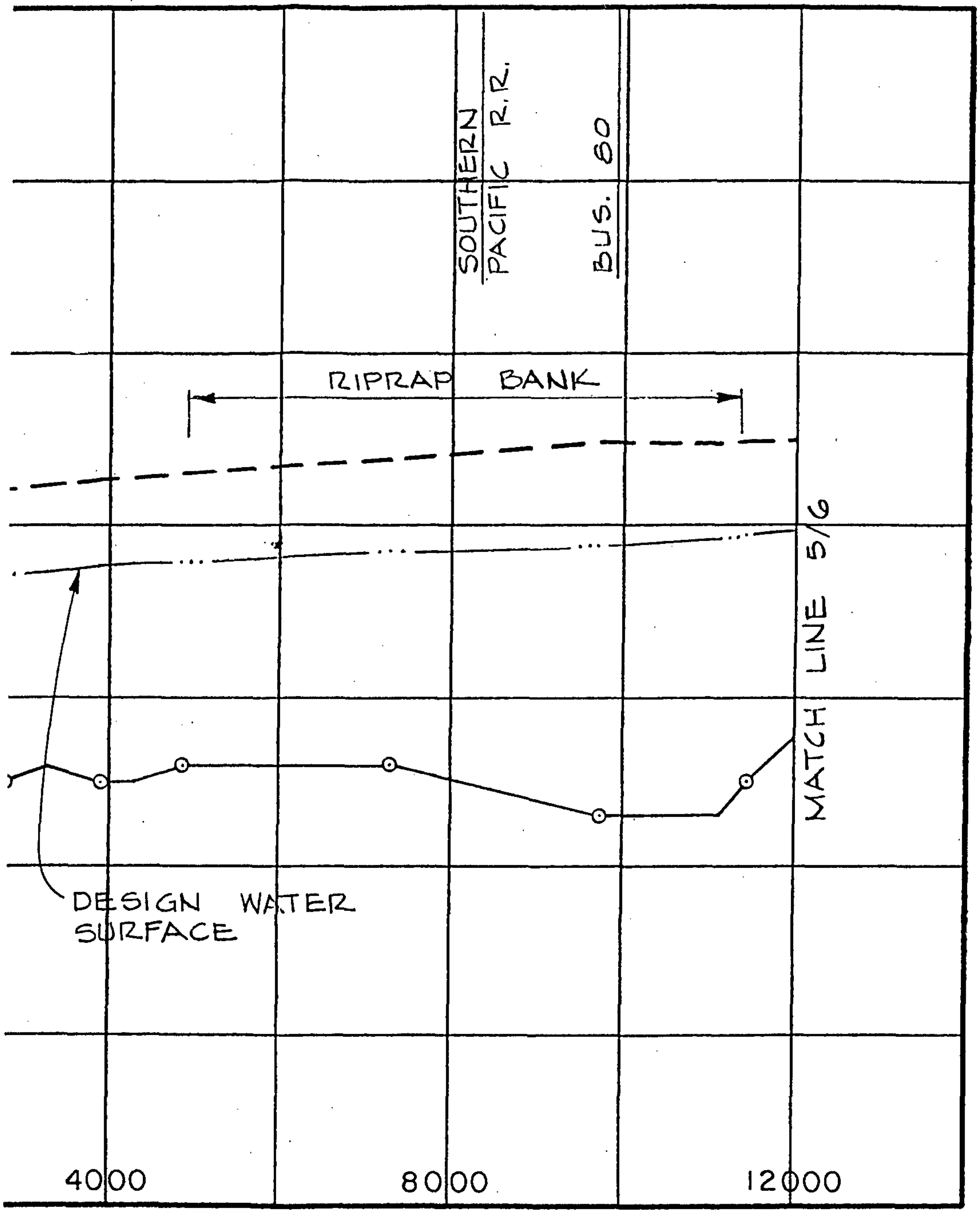


STATIONS ALONG LEVEE CENTERLINE

AMERI

LOW
NORT

SACRA



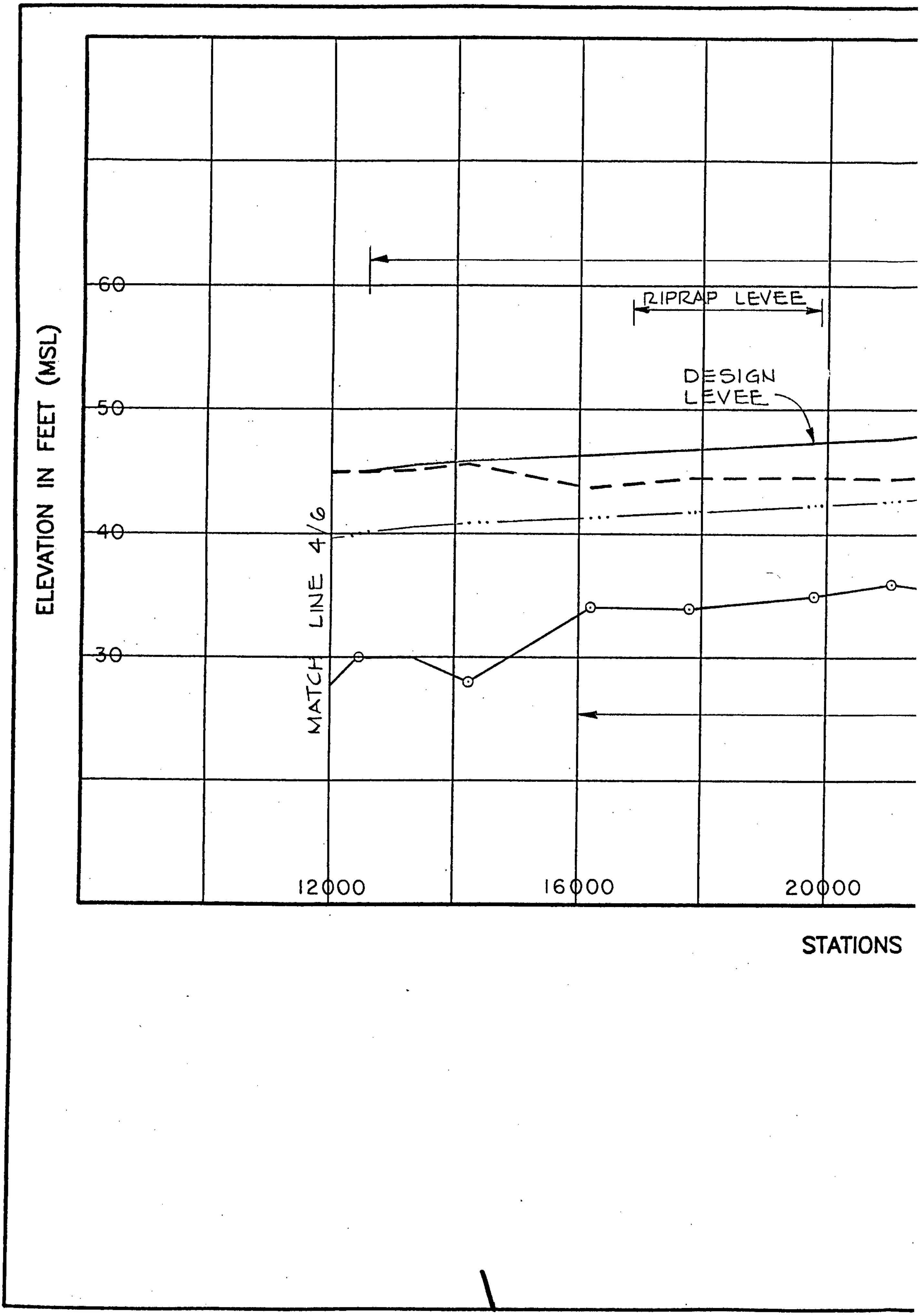
AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES
LOWER AMERICAN RIVER
NORTH (RIGHT) LEVEE PROFILE
180,000 CFS PLAN

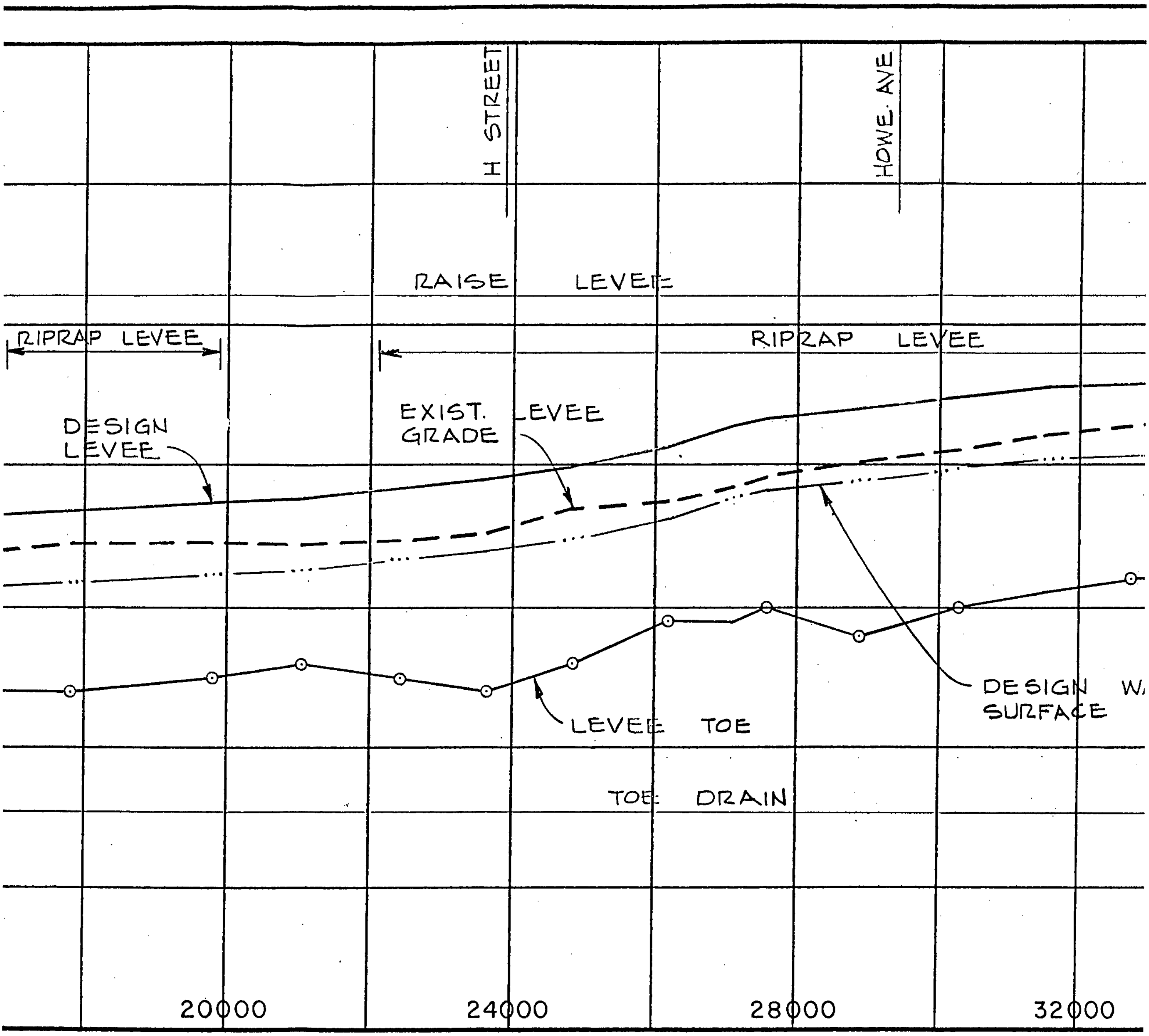
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 3 4/6

3



ELEVATION IN FEET (MSL)
60
50
40
30
RIPRAP LEVEE
DESIGN LEVEE
MATCH LINE 4/6
12000
16000
20000
STATIONS



STATIONS ALONG LEVEE CENTERLINE

AMERICA

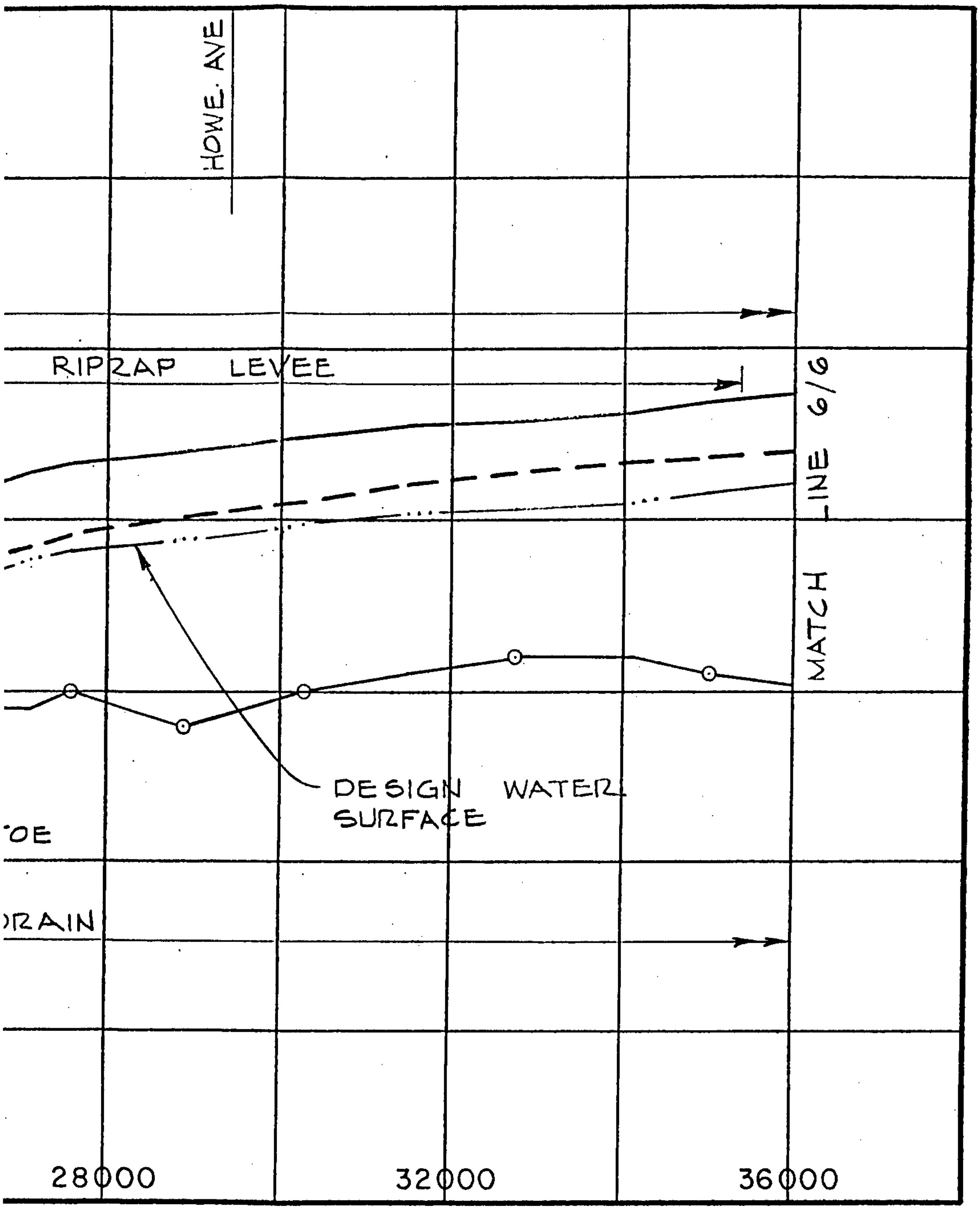
WA

LOWE

NORTH

18

SACRAMEN



AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES

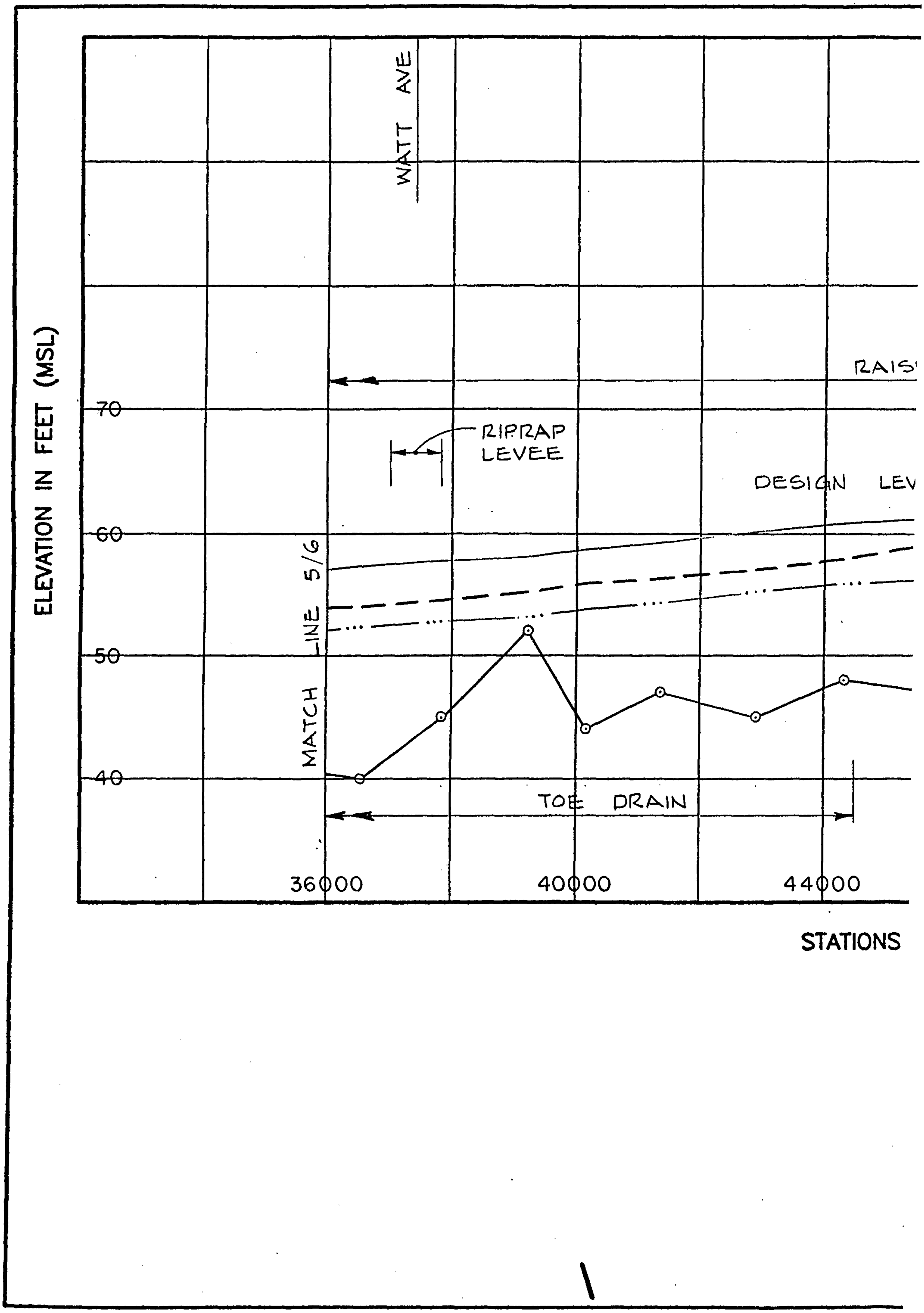
LOWER AMERICAN RIVER

NORTH (RIGHT) LEVEE PROFILE
180,000 CFS PLAN

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

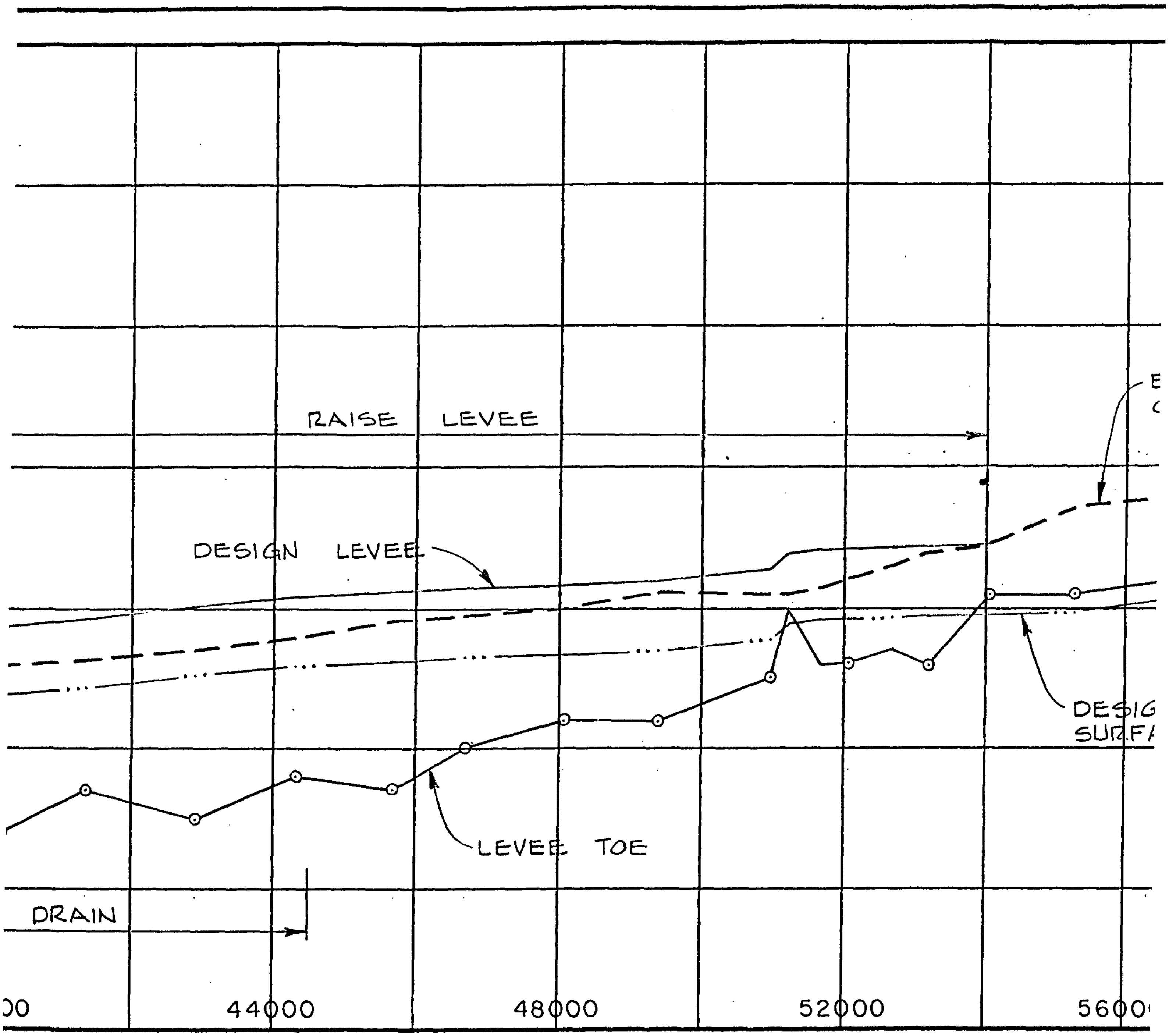
PLATE 3 5/6

3



WATT AVE
RAIS
70
RIPRAP
LEVEE
DESIGN LEV
60
MATCH LINE 5/6
50
40
TOE DRAIN
36000
40000
44000
ELEVATION IN FEET (MSL)
STATIONS

\

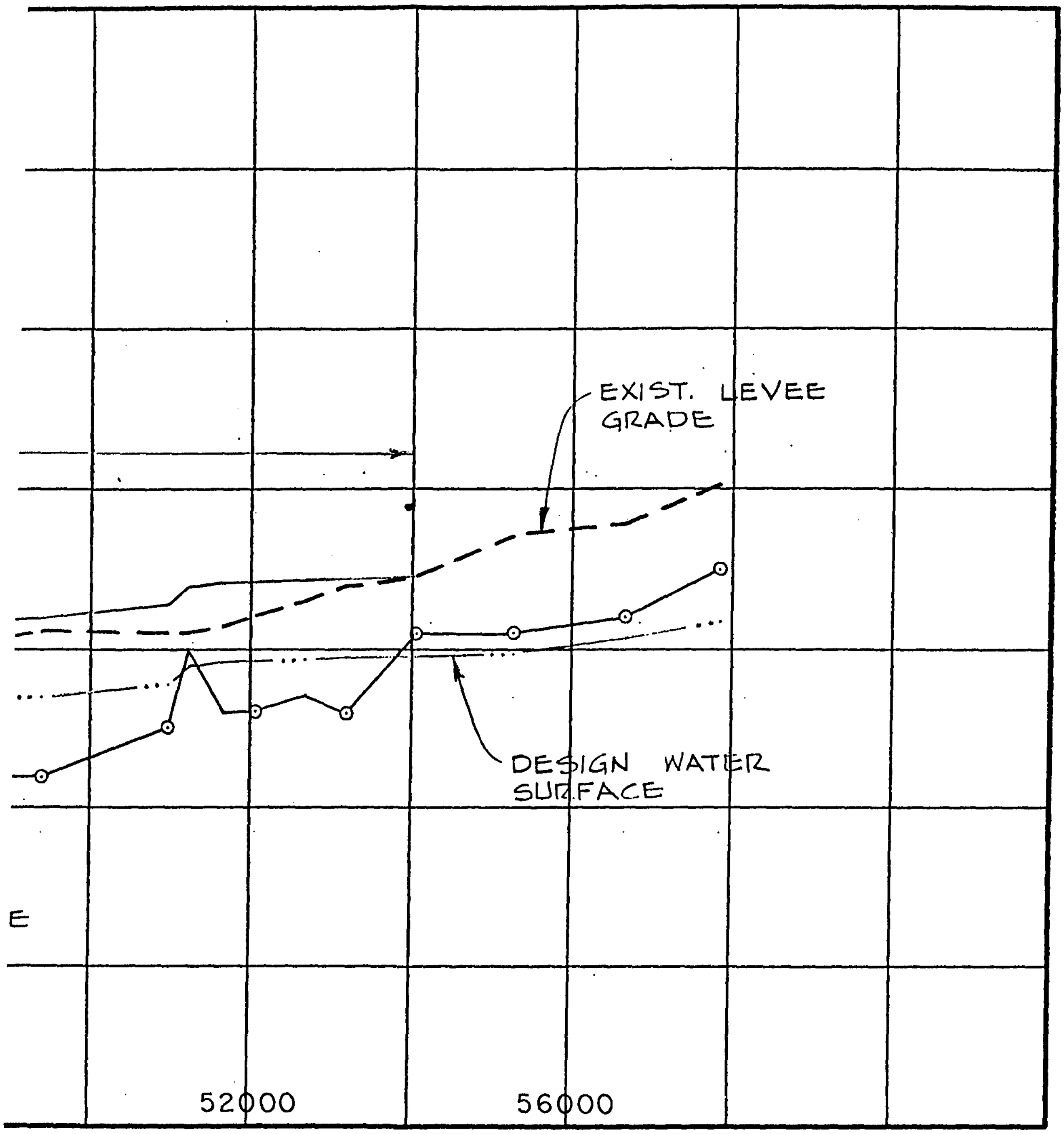


STATIONS ALONG LEVEE CENTERLINE

AME

LO'
NOR

SACI



TERLINE

AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES

LOWER AMERICAN RIVER

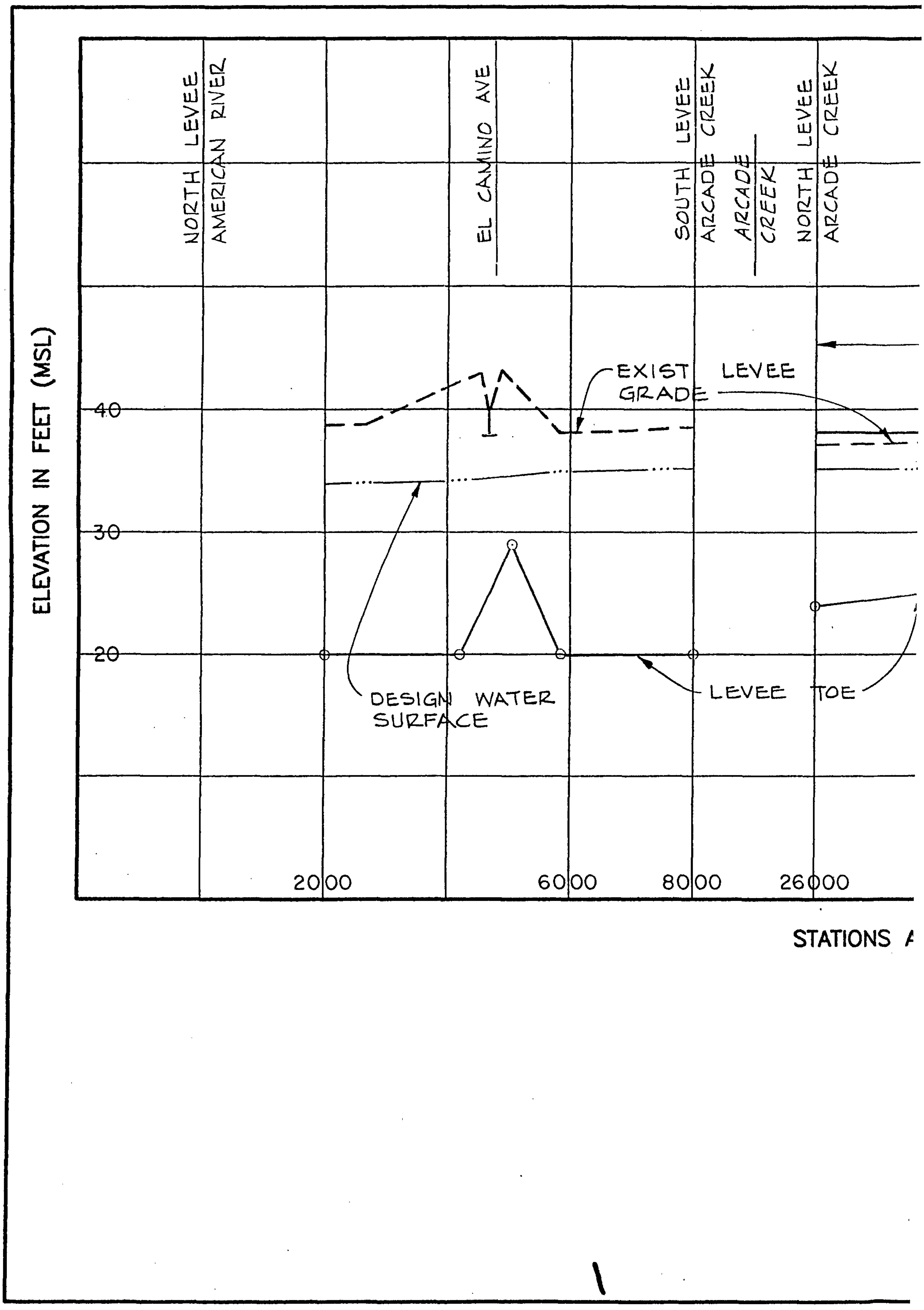
NORTH (RIGHT) LEVEE PROFILE

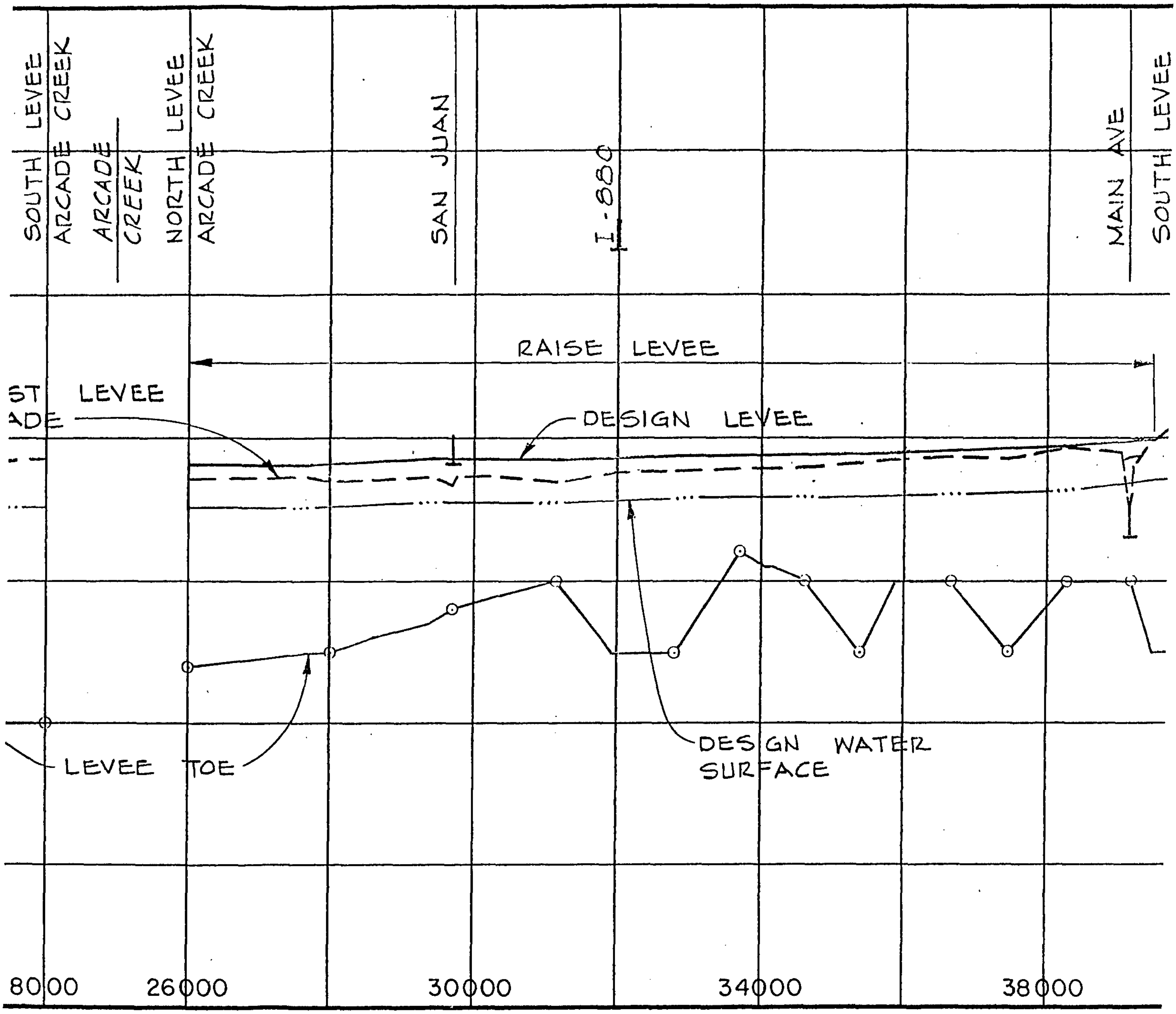
180,000 CFS PLAN

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 3 6/6

3

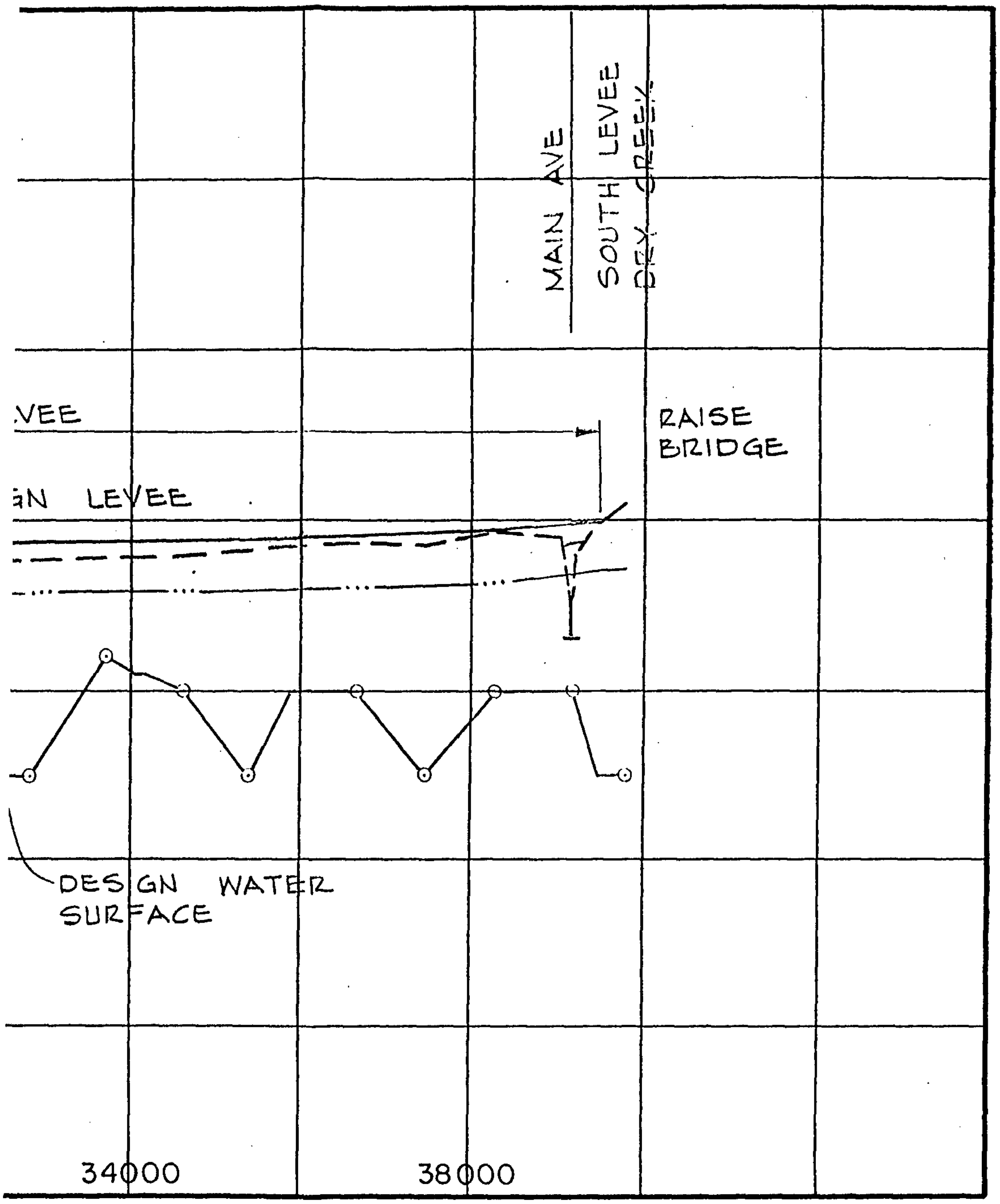

NORTH LEVEE AMERICAN RIVER
EL CAMINO AVE
SOUTH LEVEE ARCADE CREEK
ARCADE CREEK
NORTH LEVEE ARCADE CREEK
ELEVATION IN FEET (MSL)
40
30
20
EXIST LEVEE GRADE
DESIGN WATER SURFACE
LEVEE TOE
20|00
60|00
80|00
26|00
STATIONS



STATIONS ALONG LEVEE CENTERLINE

AMERICAN RIV

LE
WATER
NATOMAS
EAST (L
SE
SACRAMENTO



AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES

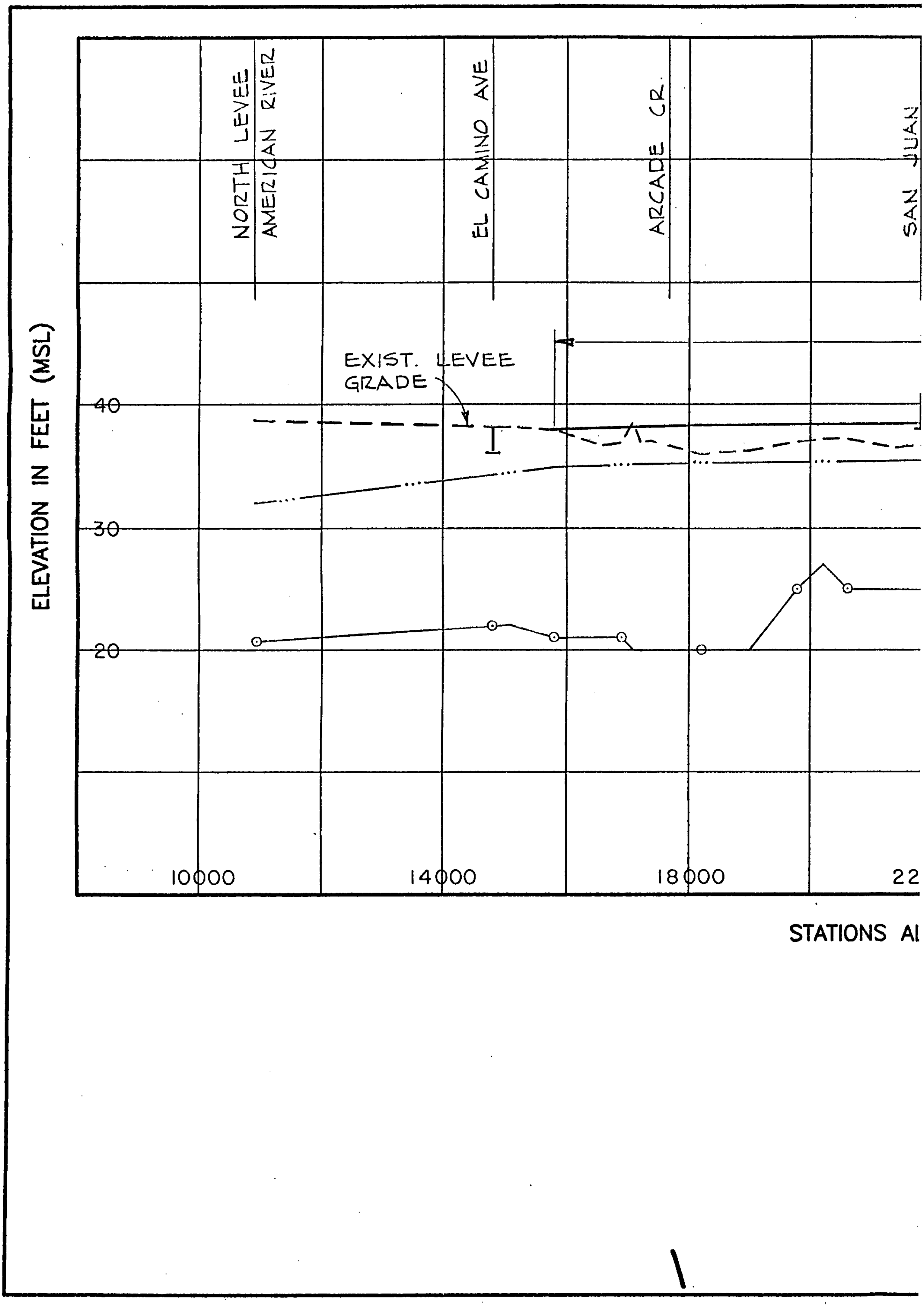
NATOMAS EAST MAIN DRAIN

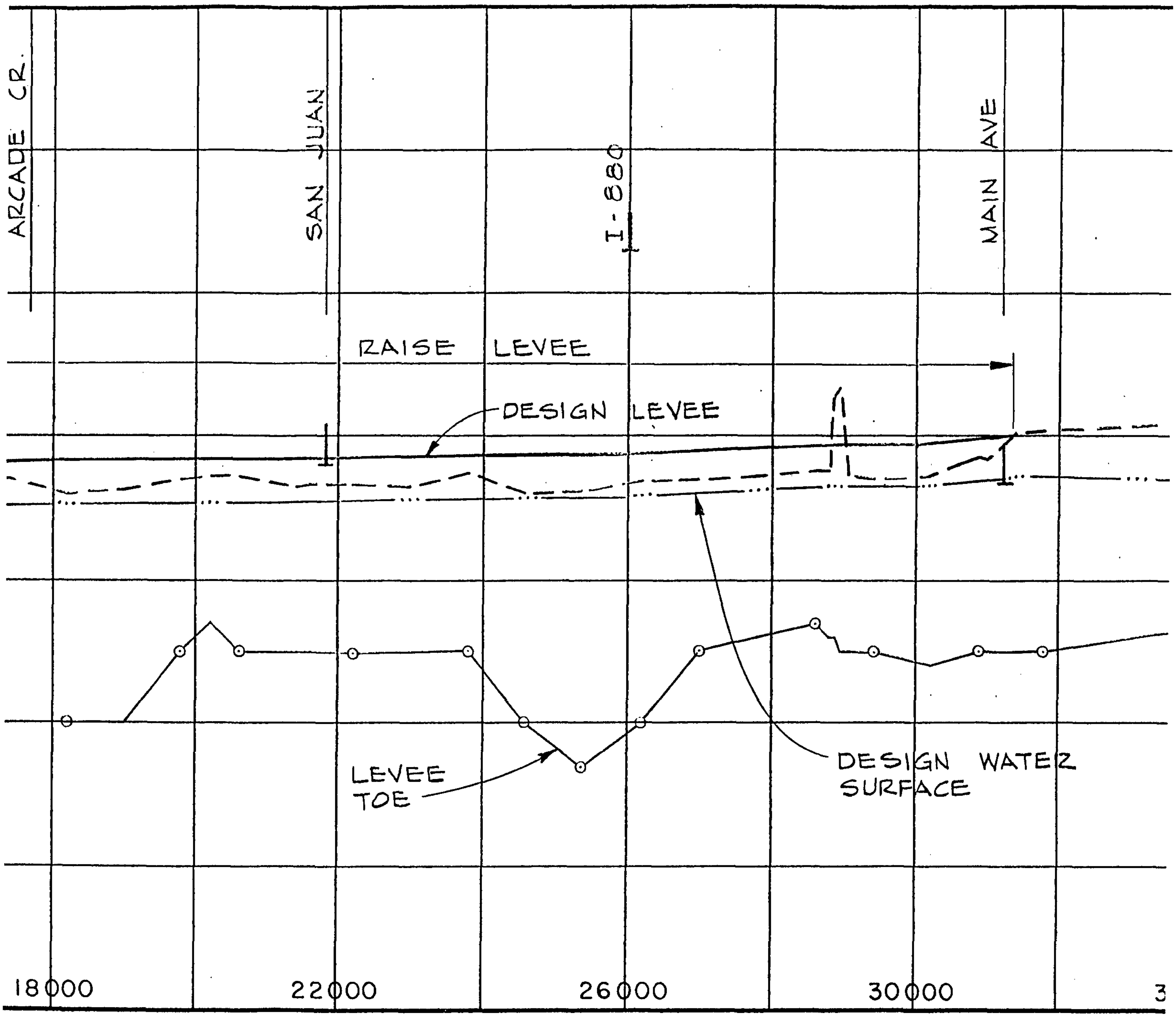
EAST (LEFT) LEVEE PROFILE
SELECTED PLAN

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 4

3

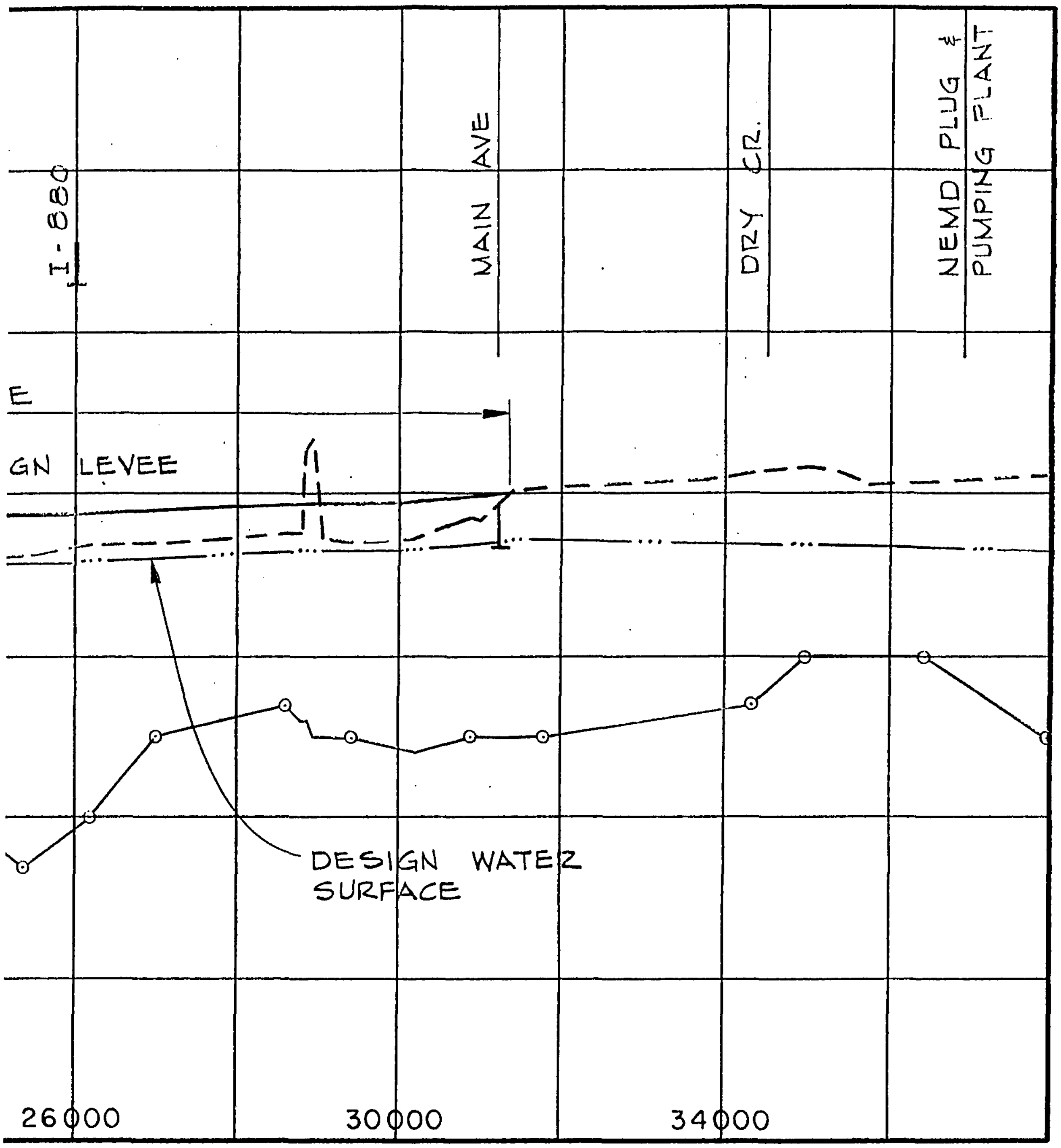

NORTH LEVEE
AMERICAN RIVER
EL CAMINO AVE
ARCADE CR.
SAN JUAN
EXIST. LEVEE
GRADE
ELEVATION IN FEET (MSL)
40
30
20
10000
14000
18000
22
STATIONS A



STATIONS ALONG LEVEE CENTERLINE

AMERICAN RI

LE
WATE
NATOMA
WEST (R
SE
SACRAMENTO



ERLINE

AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES

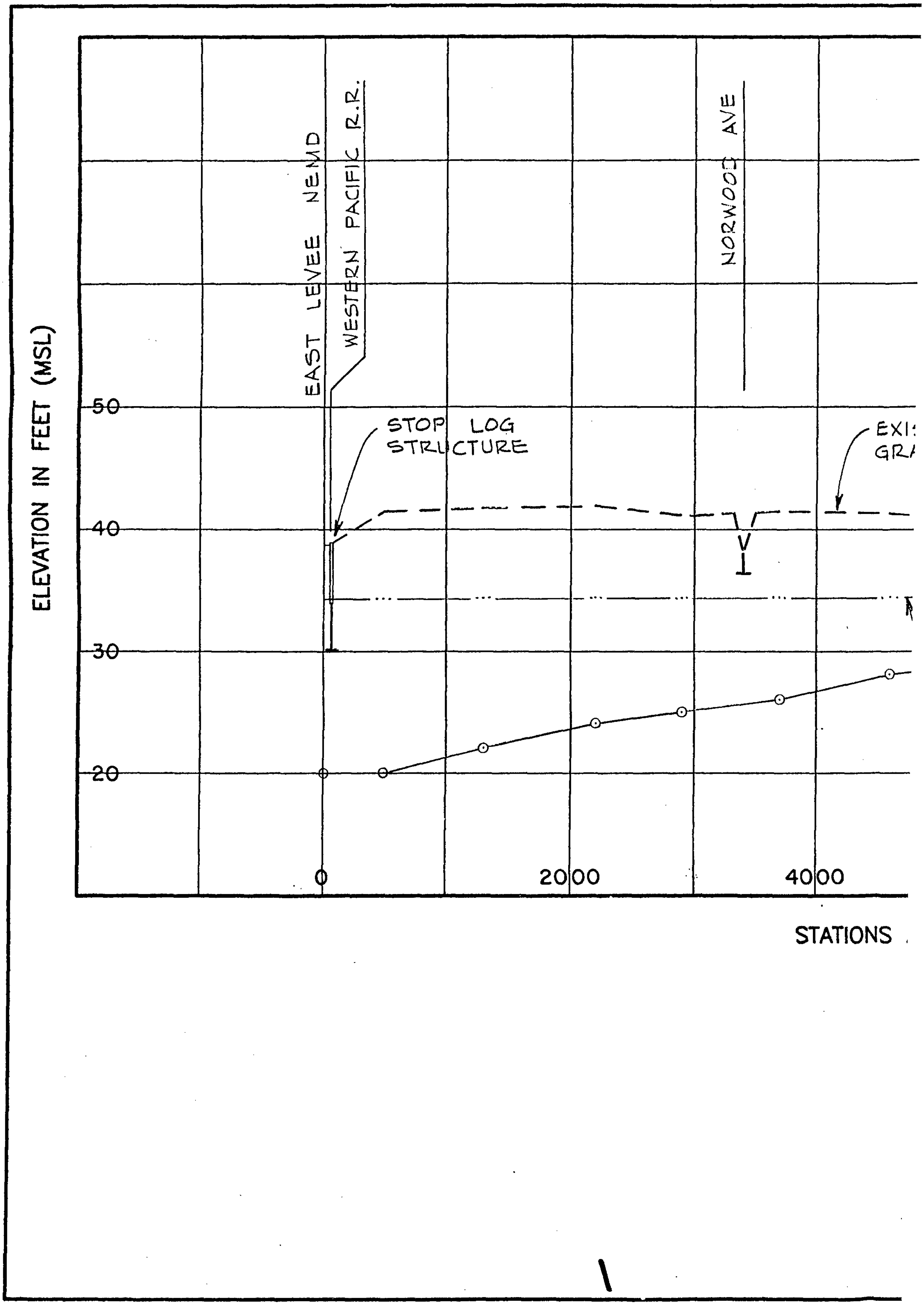
NATOMAS EAST MAIN DRAIN

WEST (RIGHT) LEVEE PROFILE

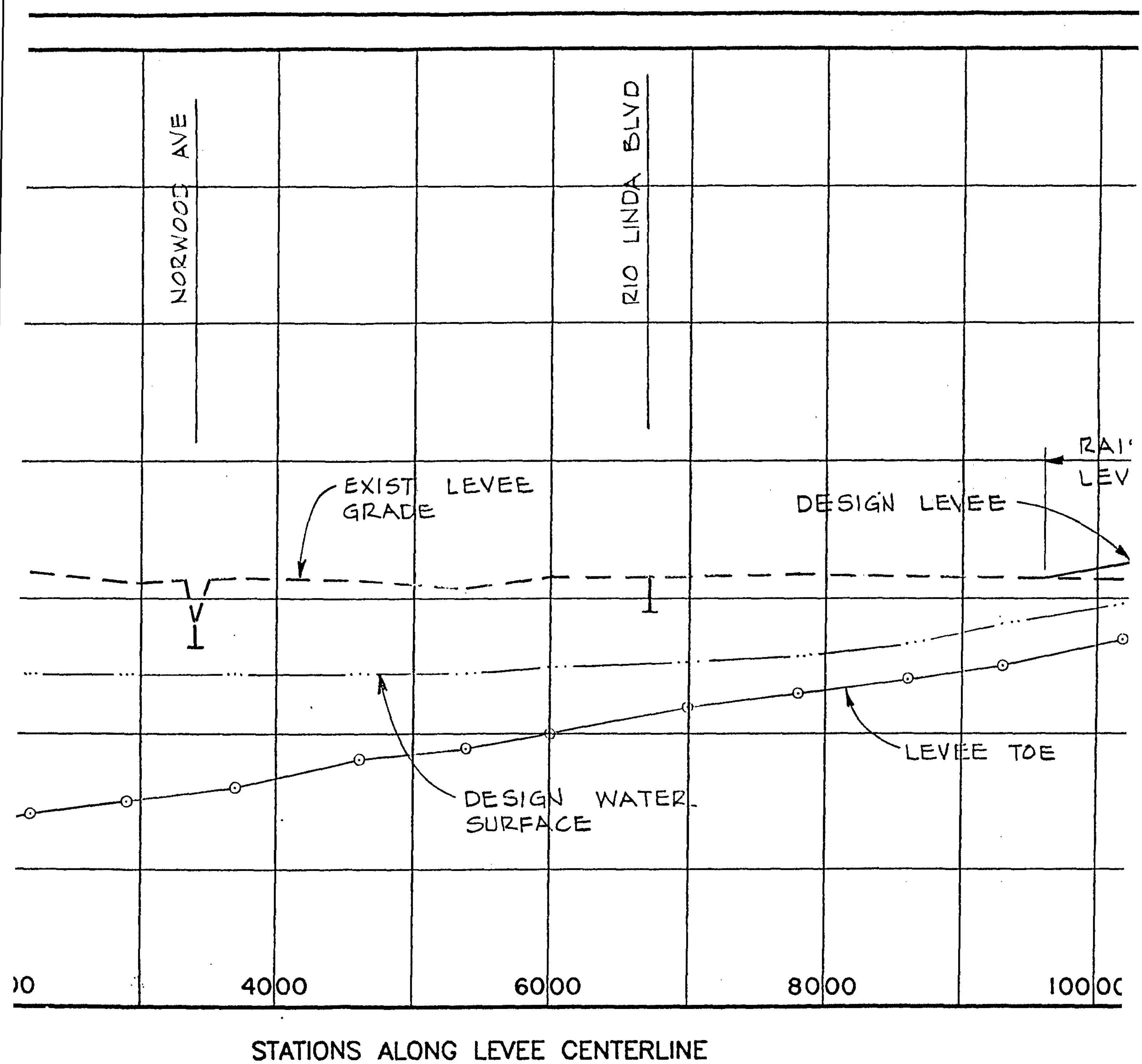
SELECTED PLAN

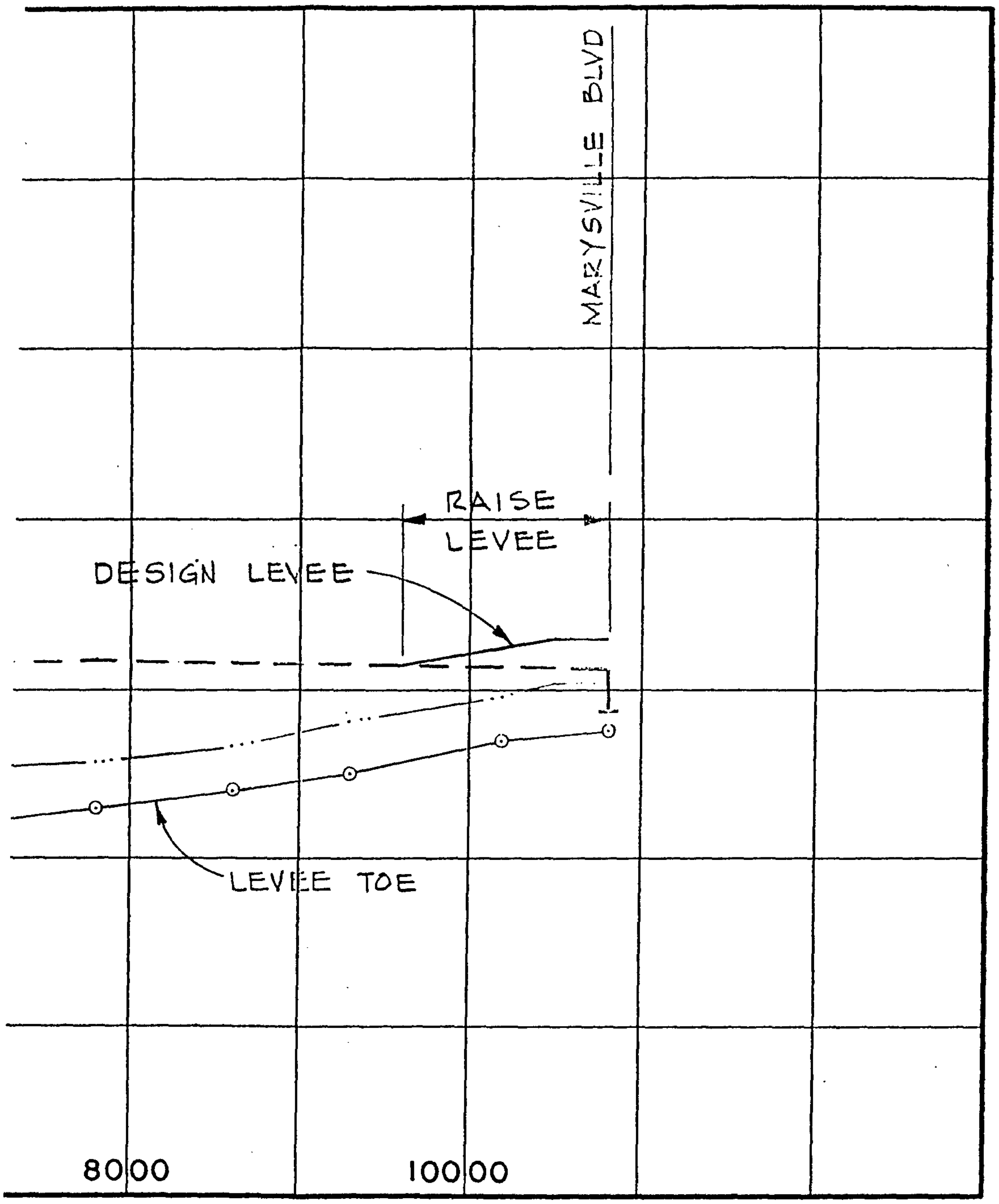
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 5



ELEVATION IN FEET (MSL)
50
40
30
20
EAST LEVEE NEMD
WESTERN PACIFIC R.R.
NORWOOD AVE
STOP LOG STRUCTURE
EXI
GRA
0
2000
4000
STATIONS


NORWOOD AVE
RIO LINDA BLVD
EXIST LEVEE GRADE
DESIGN LEVEE
RAI
LEV
DESIGN WATER SURFACE
LEVEE TOE
)0
4000
6000
8000
10000
STATIONS ALONG LEVEE CENTERLINE
AME
SOL
SACI



AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES

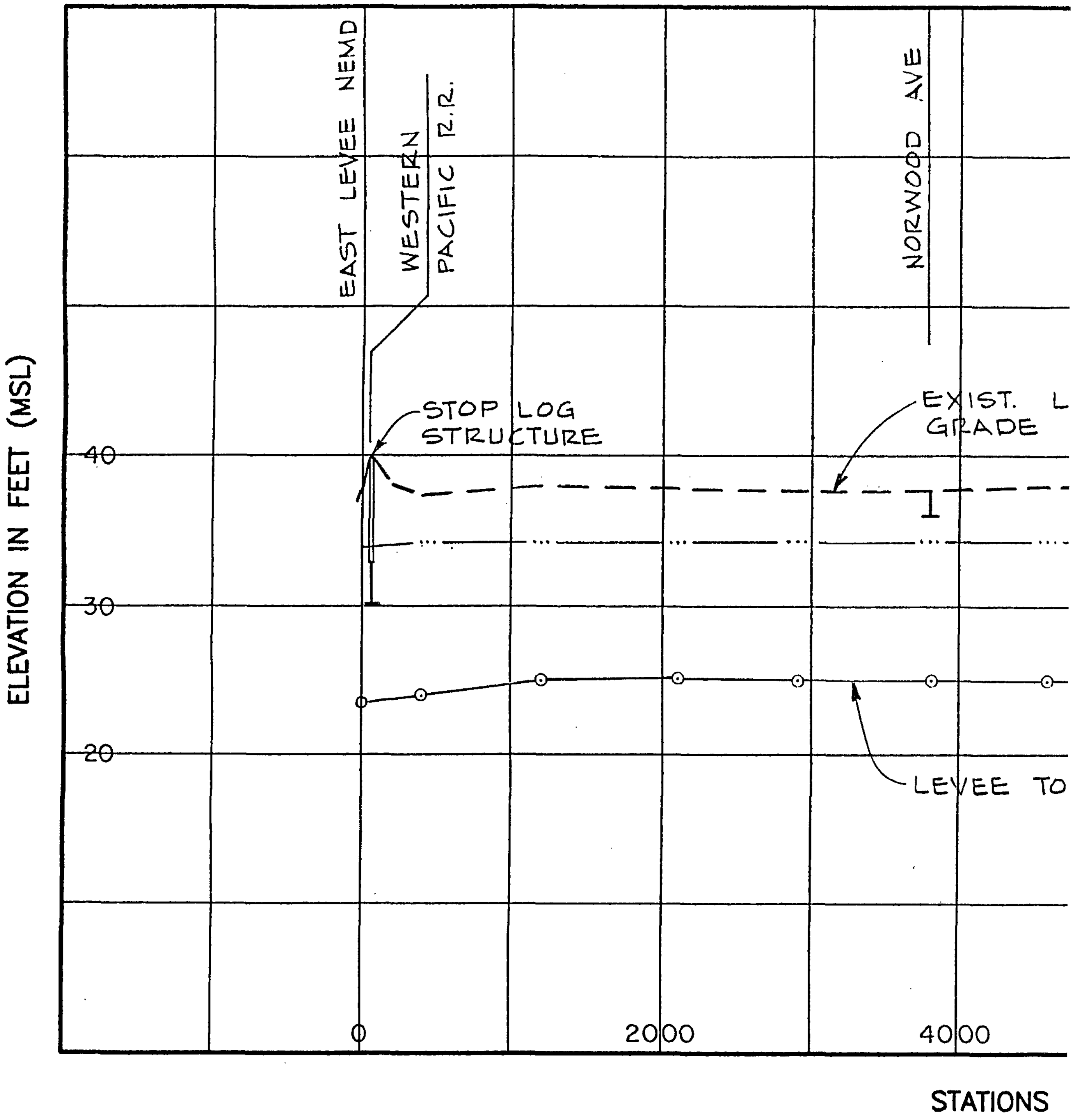
ARCADE CREEK

SOUTH (LEFT) LEVEE PROFILE
SELECTED PLAN

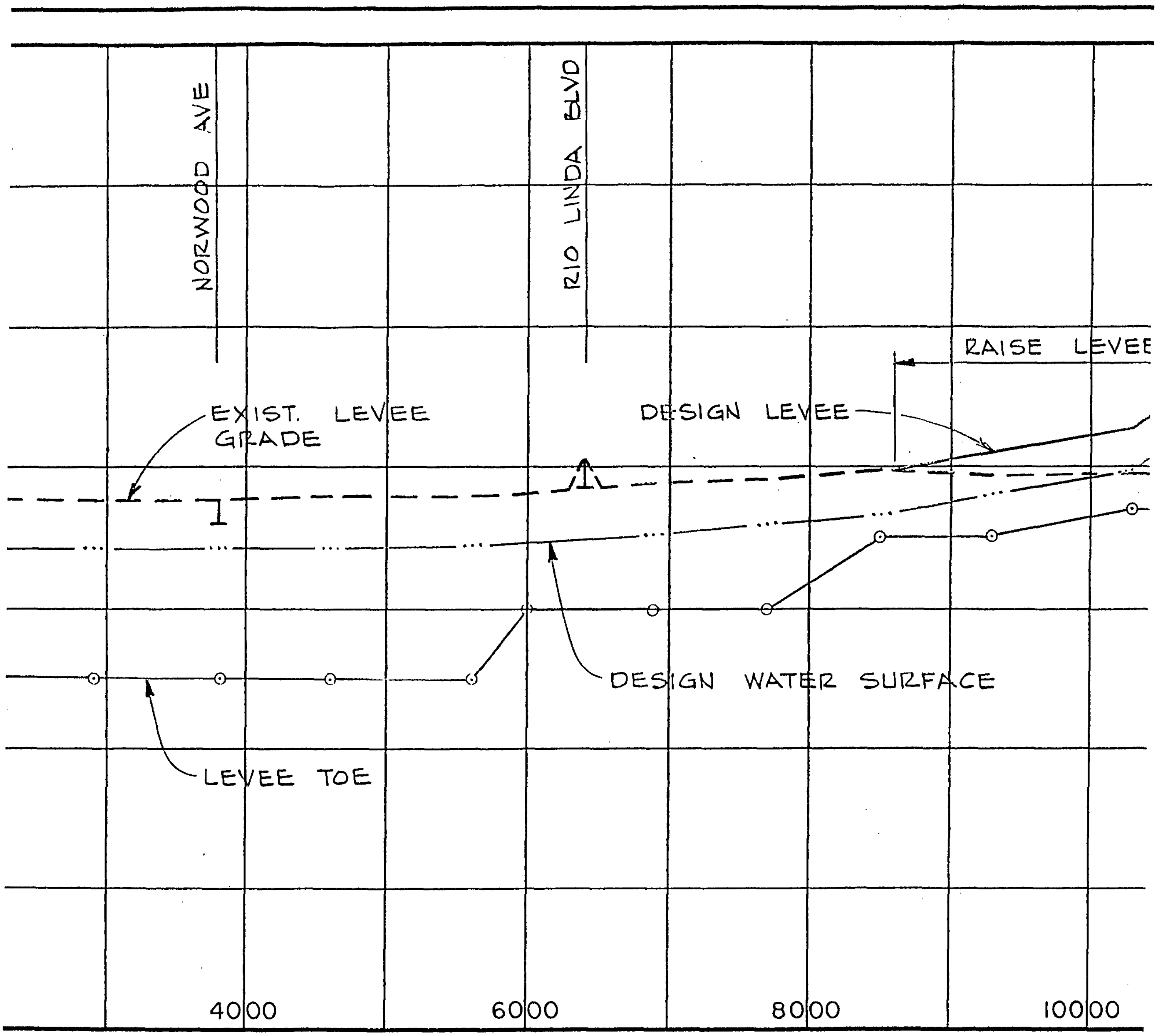
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 6

3



ELEVATION IN FEET (MSL)
40
30
20
EAST LEVEE NEMD
WESTERN PACIFIC R.R.
NORWOOD AVE
STOP LOG STRUCTURE
EXIST. L
GRADE
LEVEE TO
0
2000
4000
STATIONS



STATIONS ALONG LEVEE CENTERLINE

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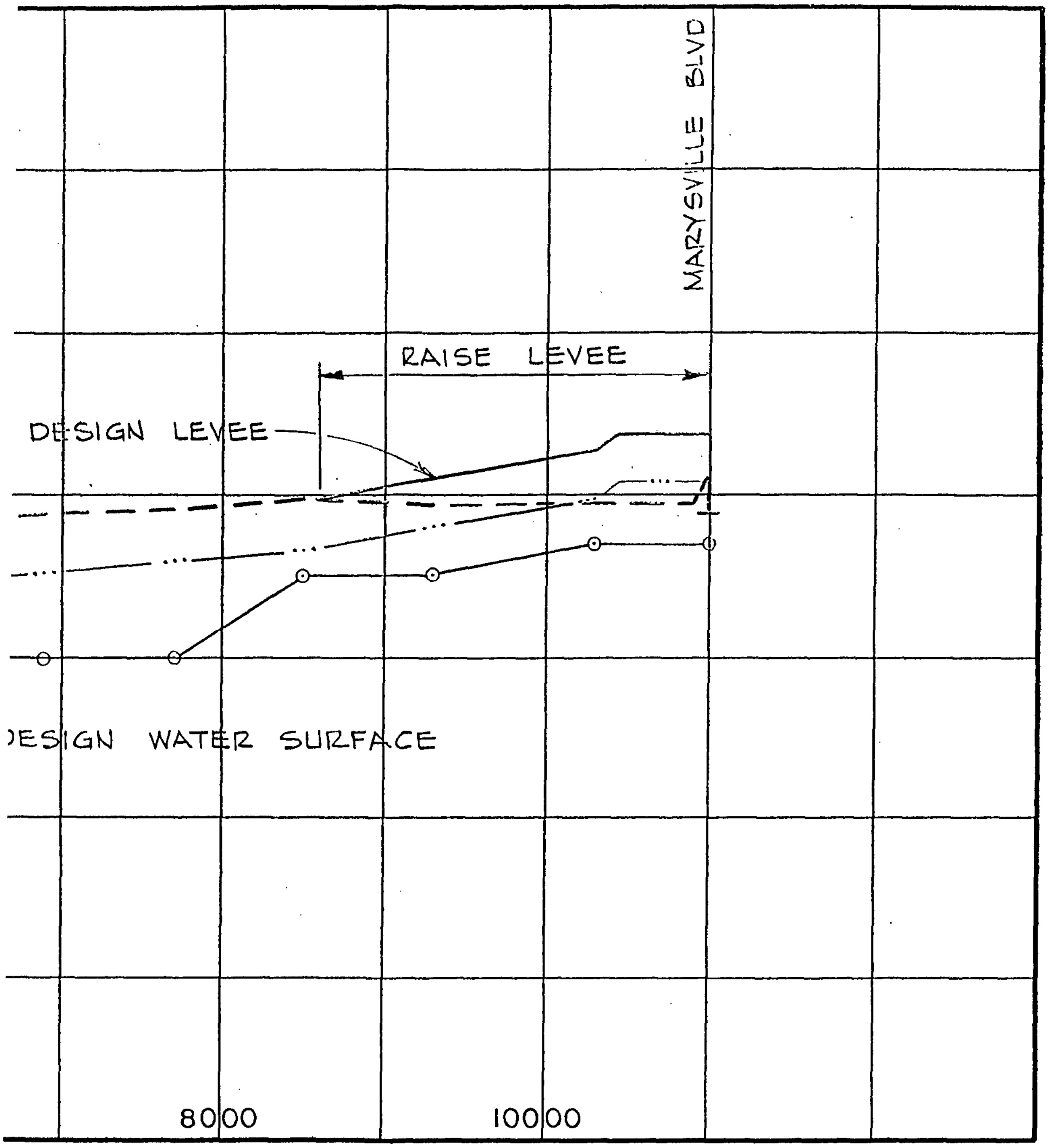

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AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES

ARCADE CREEK

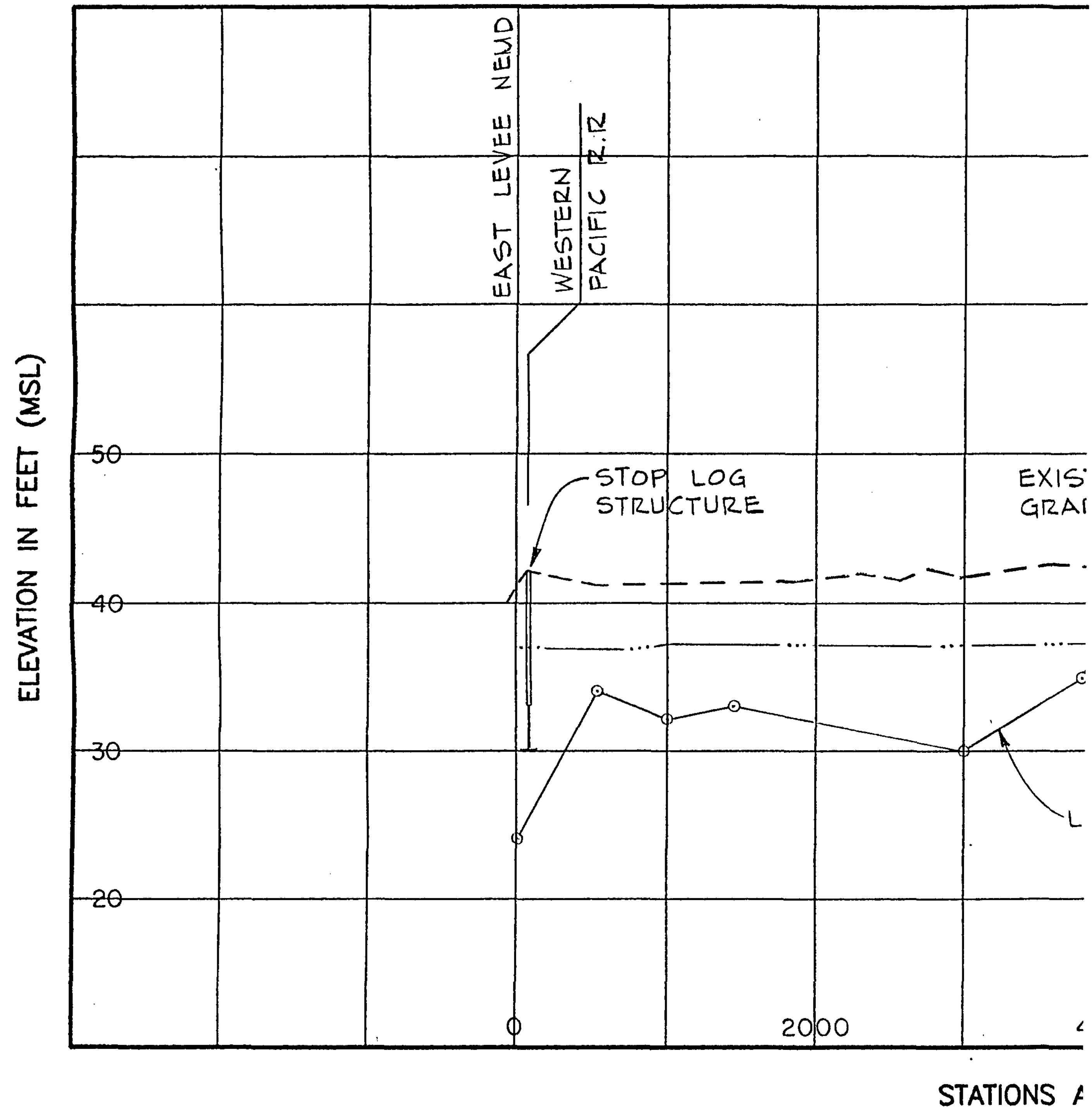
NORTH (RIGHT) LEVEE PROFILE

SELECTED PLAN

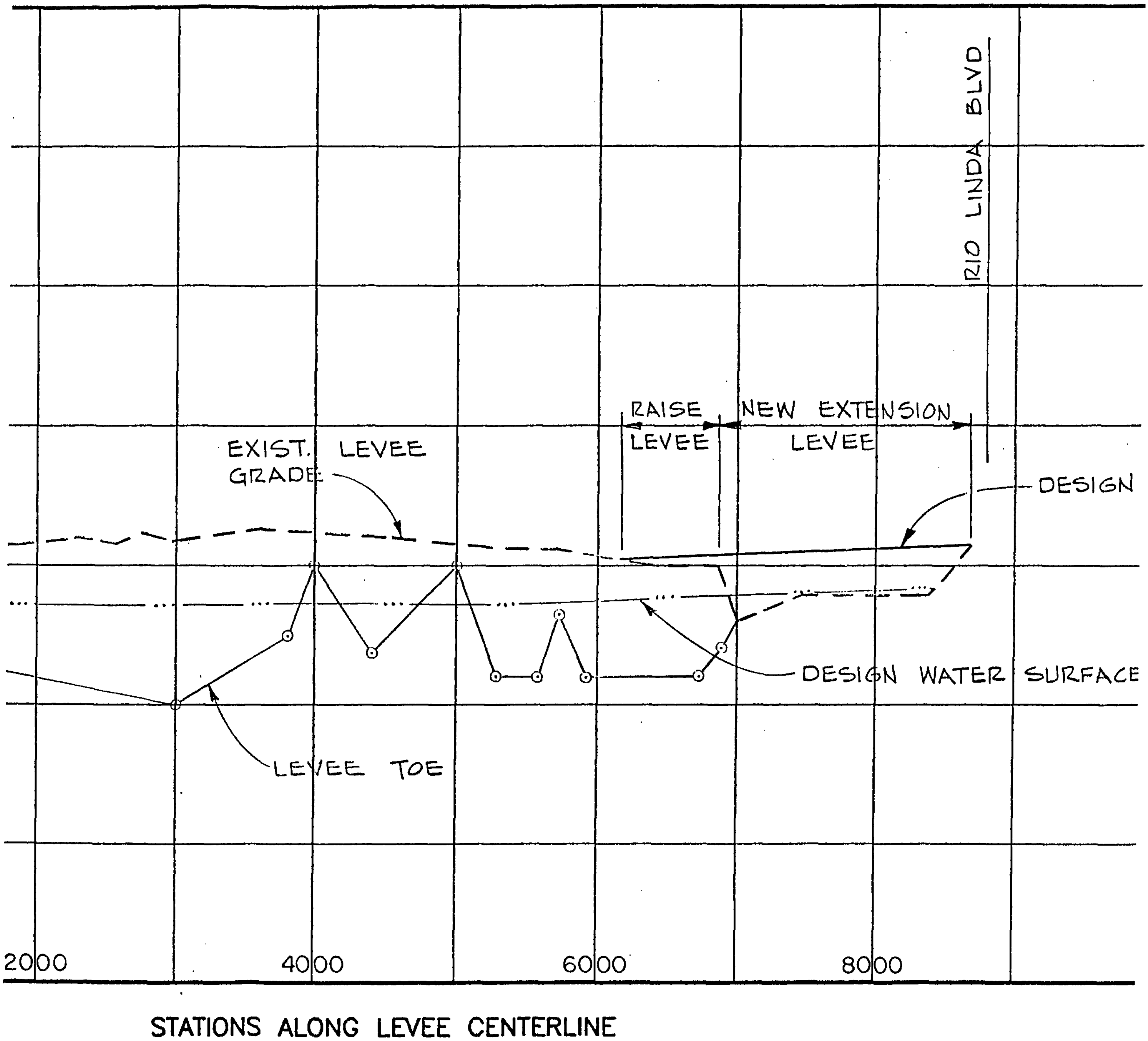
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 7

3



ELEVATION IN FEET (MSL)
EAST LEVEE NEMD
WESTERN
PACIFIC R.R
STOP LOG
STRUCTURE
EXIS
GRA
L
50
40
30
20
0
2000
STATIONS



AMERICAN RIVE

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WATER

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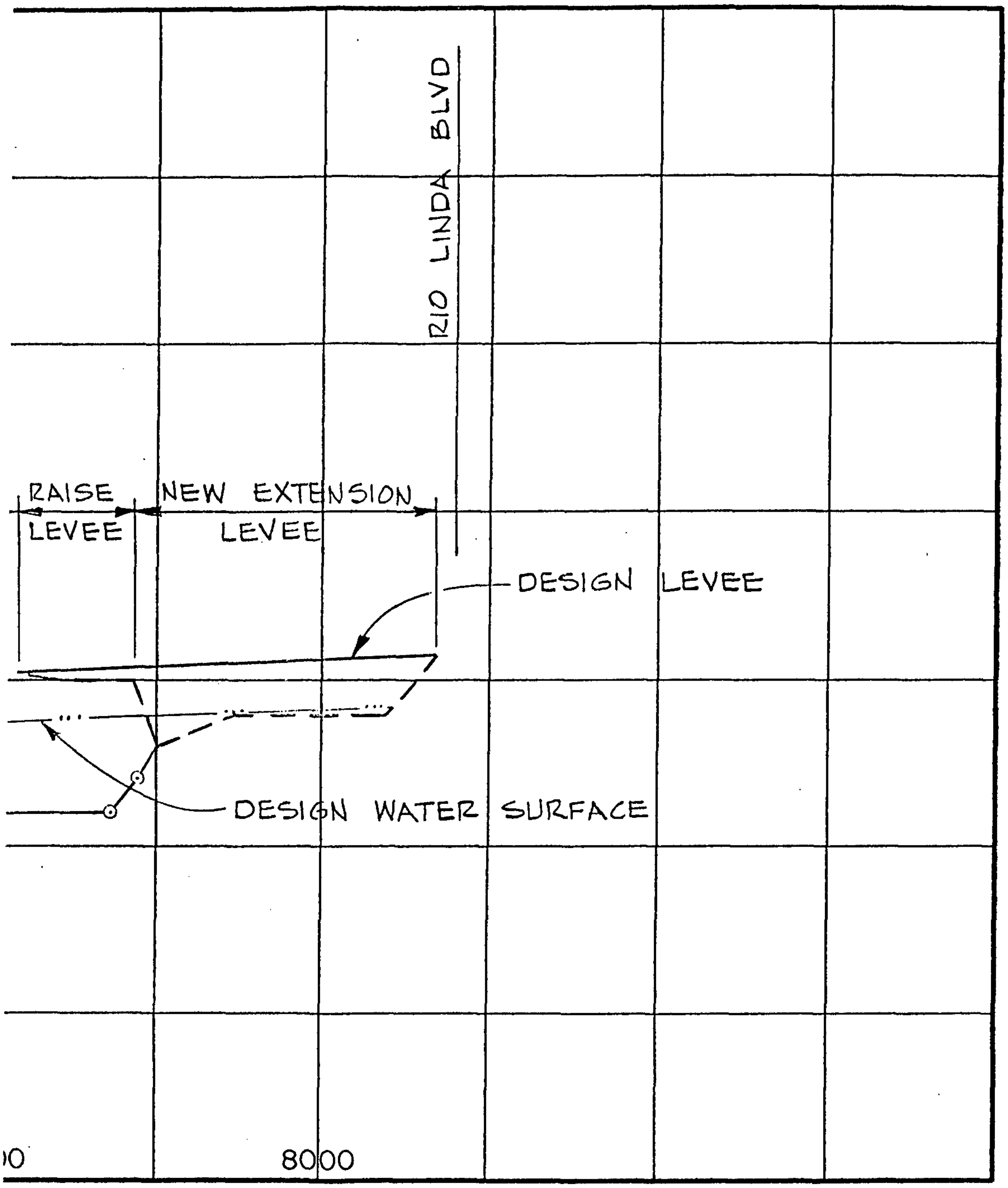
SOUTH (L

SEI

SACRAMENTO DI



AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES

DRY CREEK

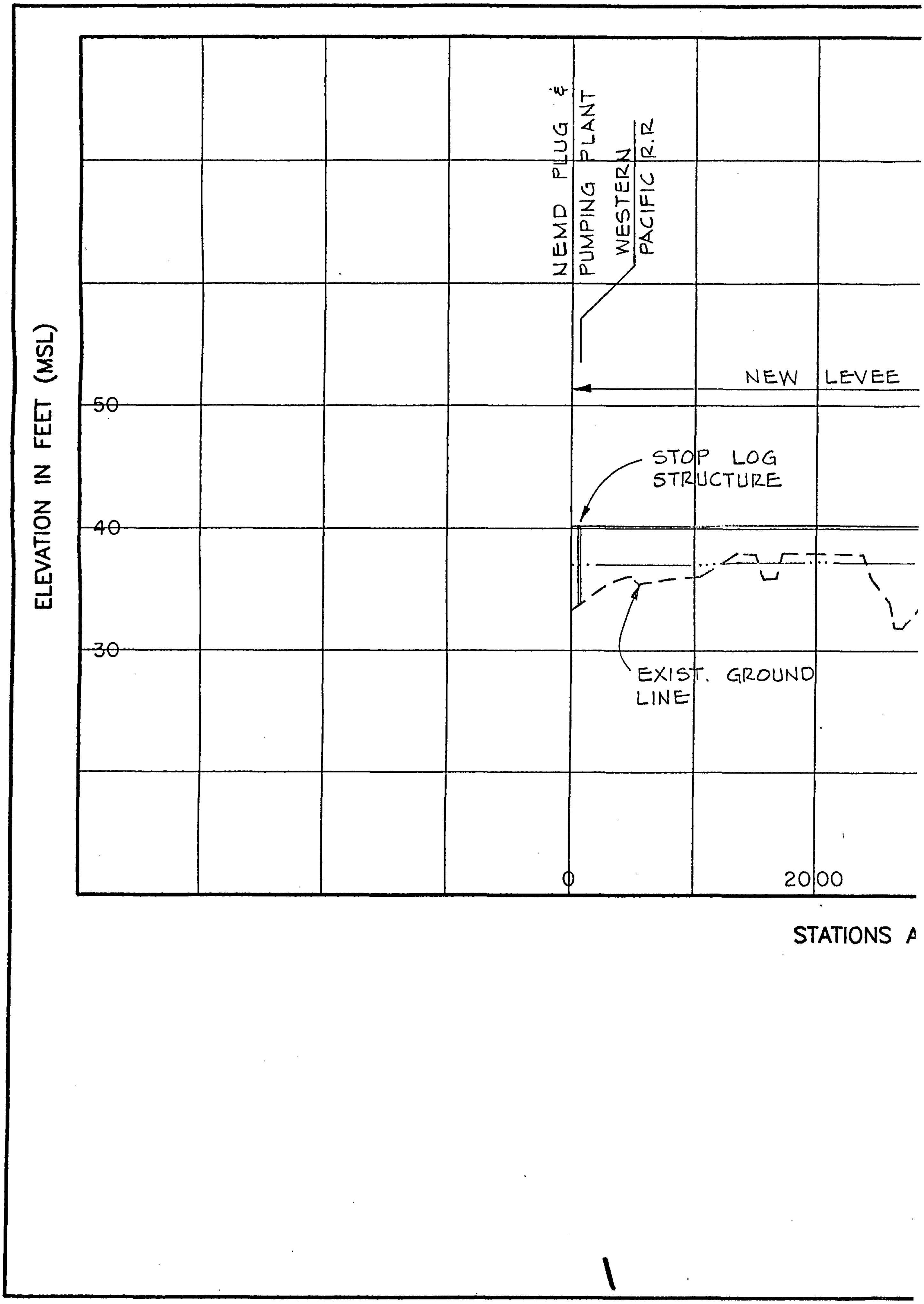
SOUTH (LEFT) LEVEE PROFILE

SELECTED PLAN

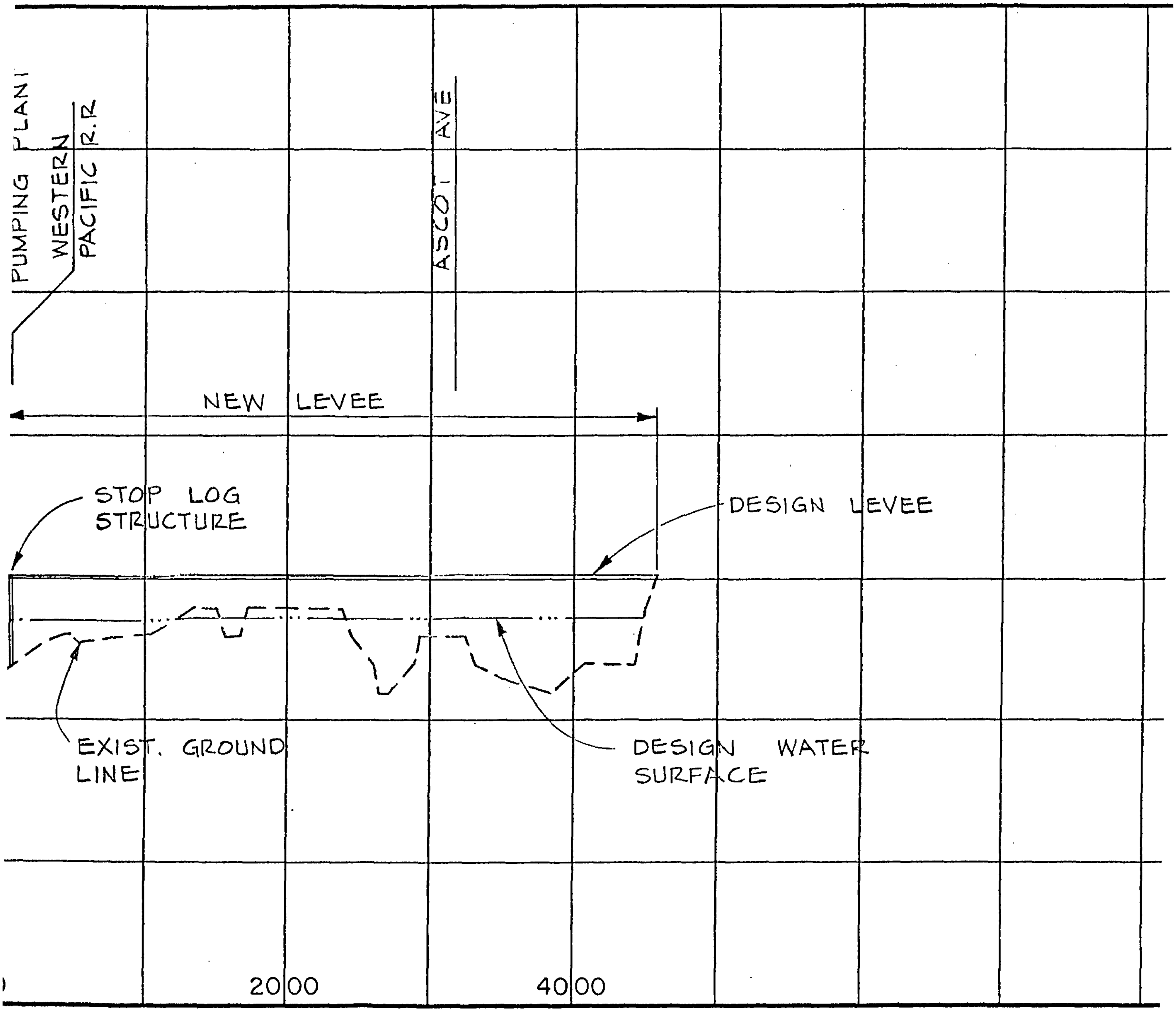
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 8

3



NEMD PLUG ¢
PUMPING PLANT
WESTERN PACIFIC R.R
NEW LEVEE
STOP LOG STRUCTURE
EXIST. GROUND LINE
ELEVATION IN FEET (MSL)
50
40
30
0
20|00
STATIONS A

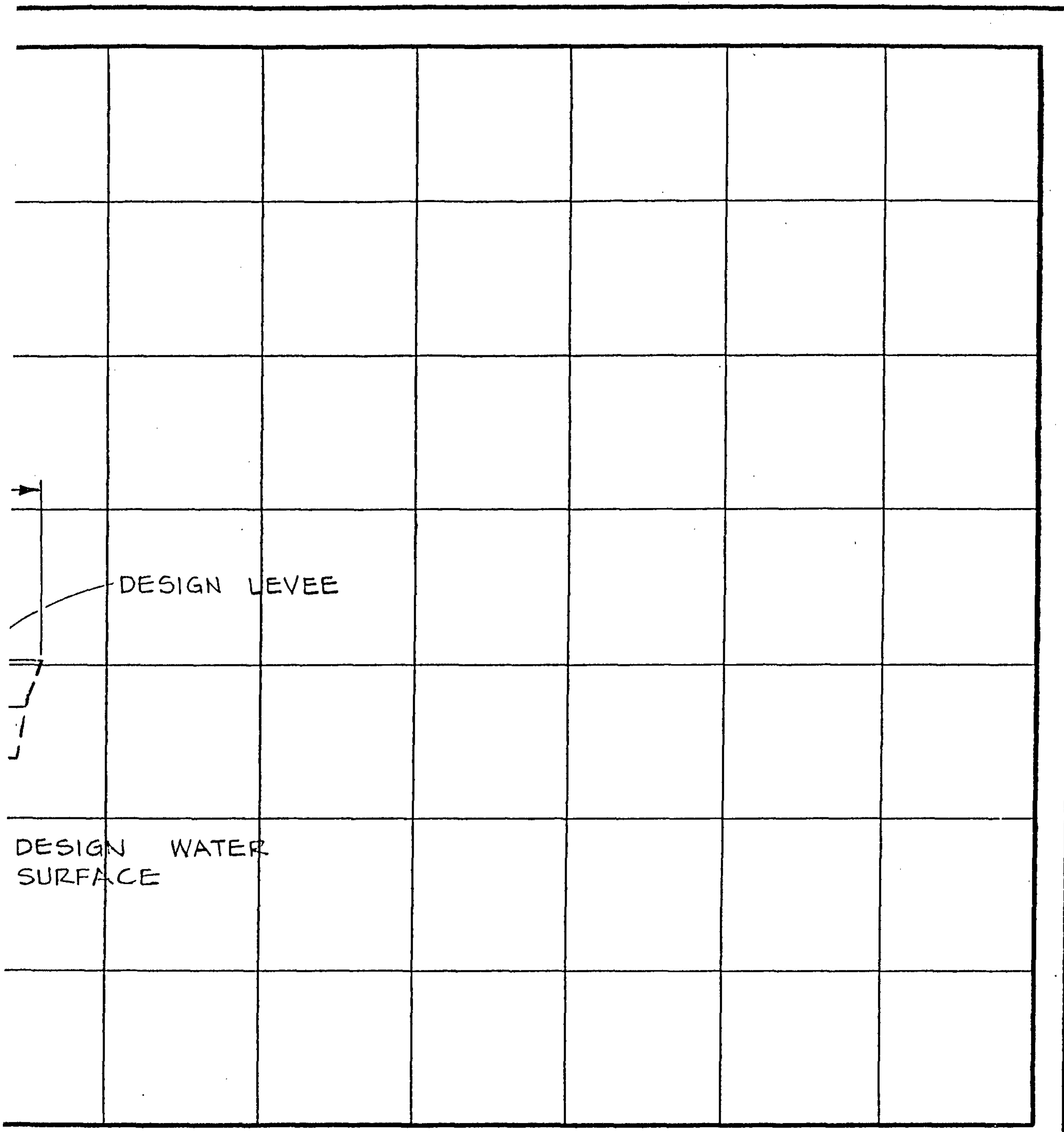


STATIONS ALONG LEVEE CENTERLINE

AM

NO

SA



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AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES

DRY CREEK

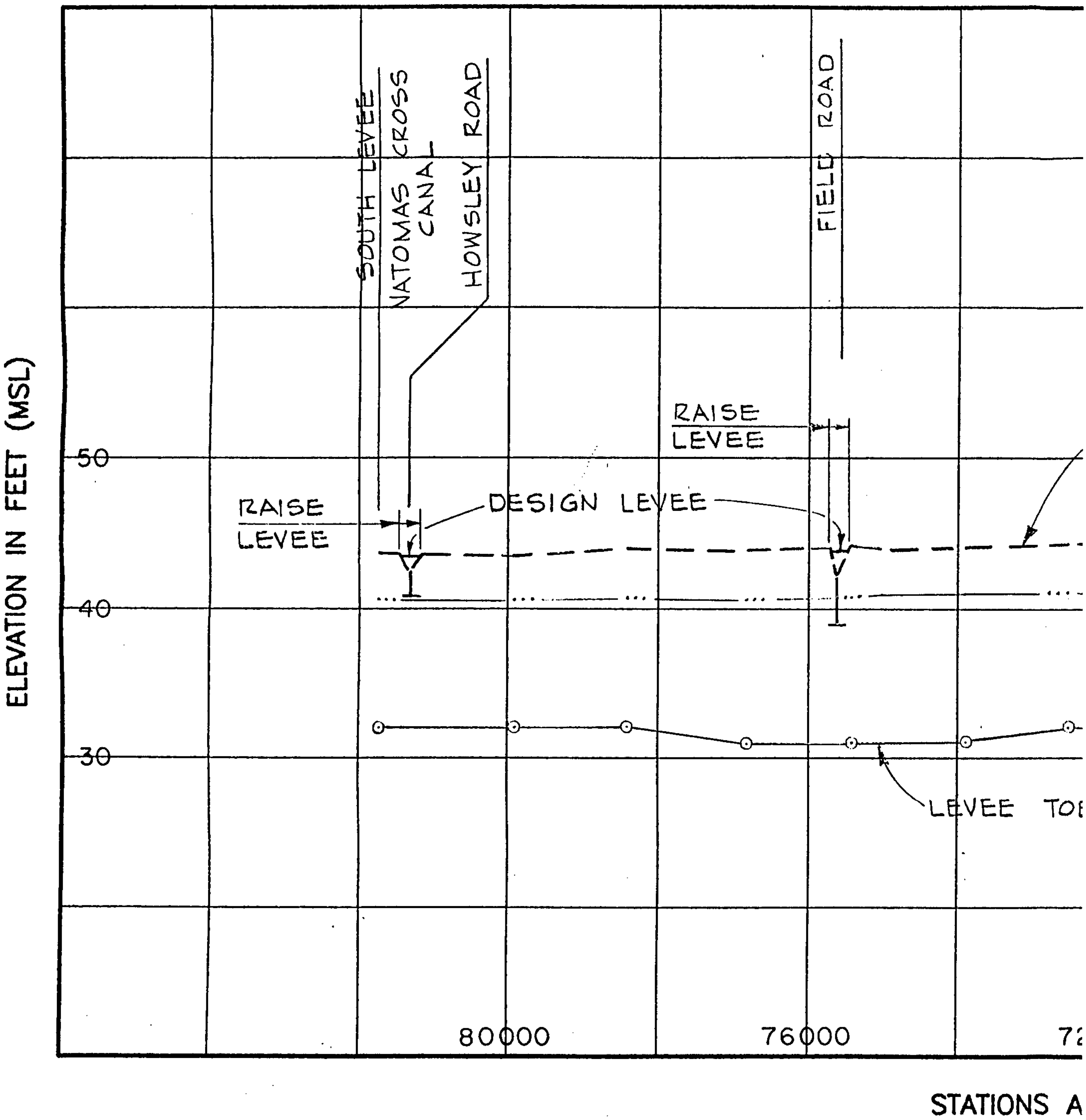
NORTH (RIGHT) LEVEE PROFILE

SELECTED PLAN

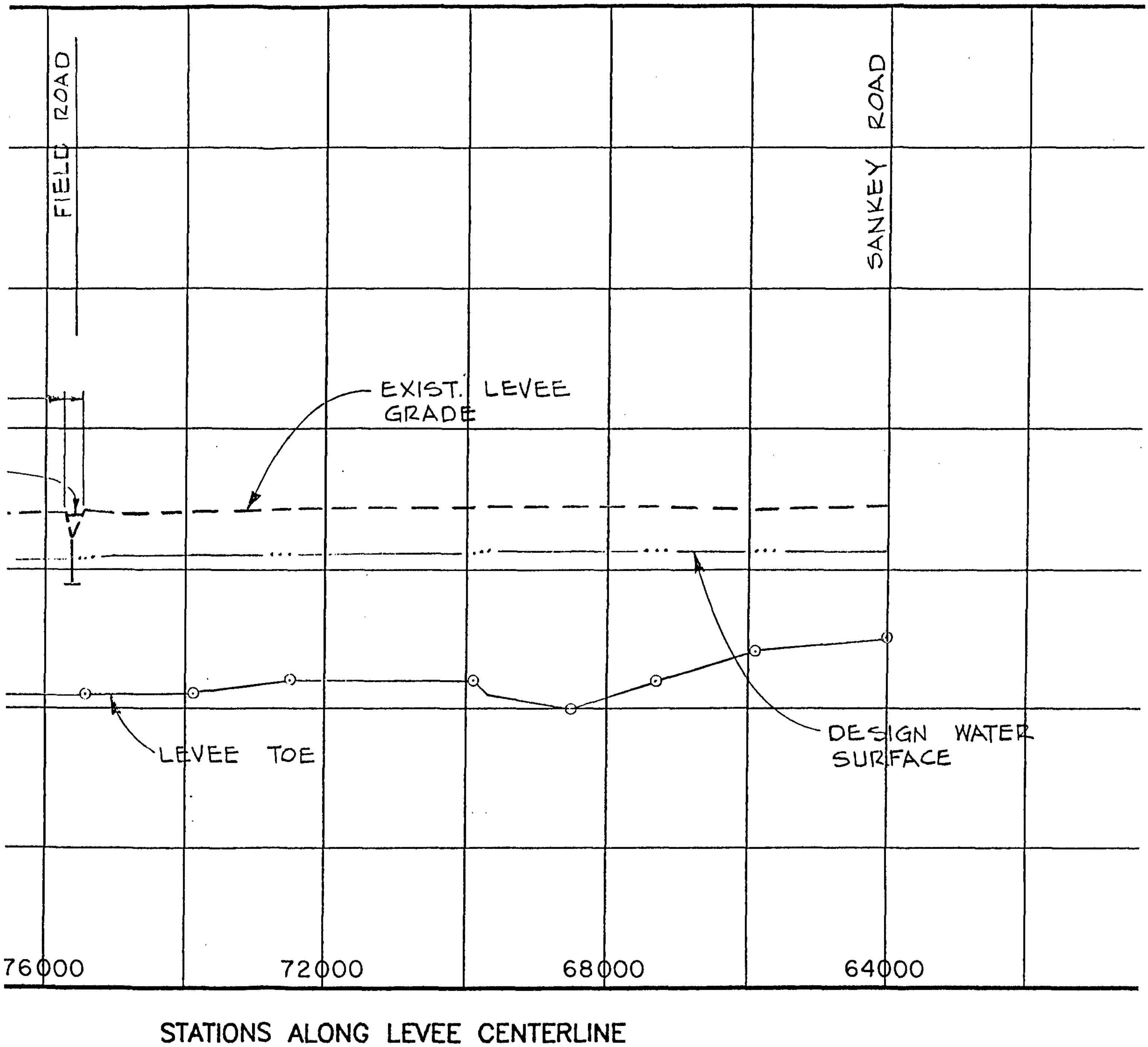
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 9

3



SOUTH LEVEE
NATOMAS CROSS CANAL
HOWSLEY ROAD
FIELD ROAD
RAISE LEVEE
RAISE LEVEE
DESIGN LEVEE
LEVEE TOE
ELEVATION IN FEET (MSL)
50
40
30
80000
76000
72
STATIONS A



AMERICAN RIV

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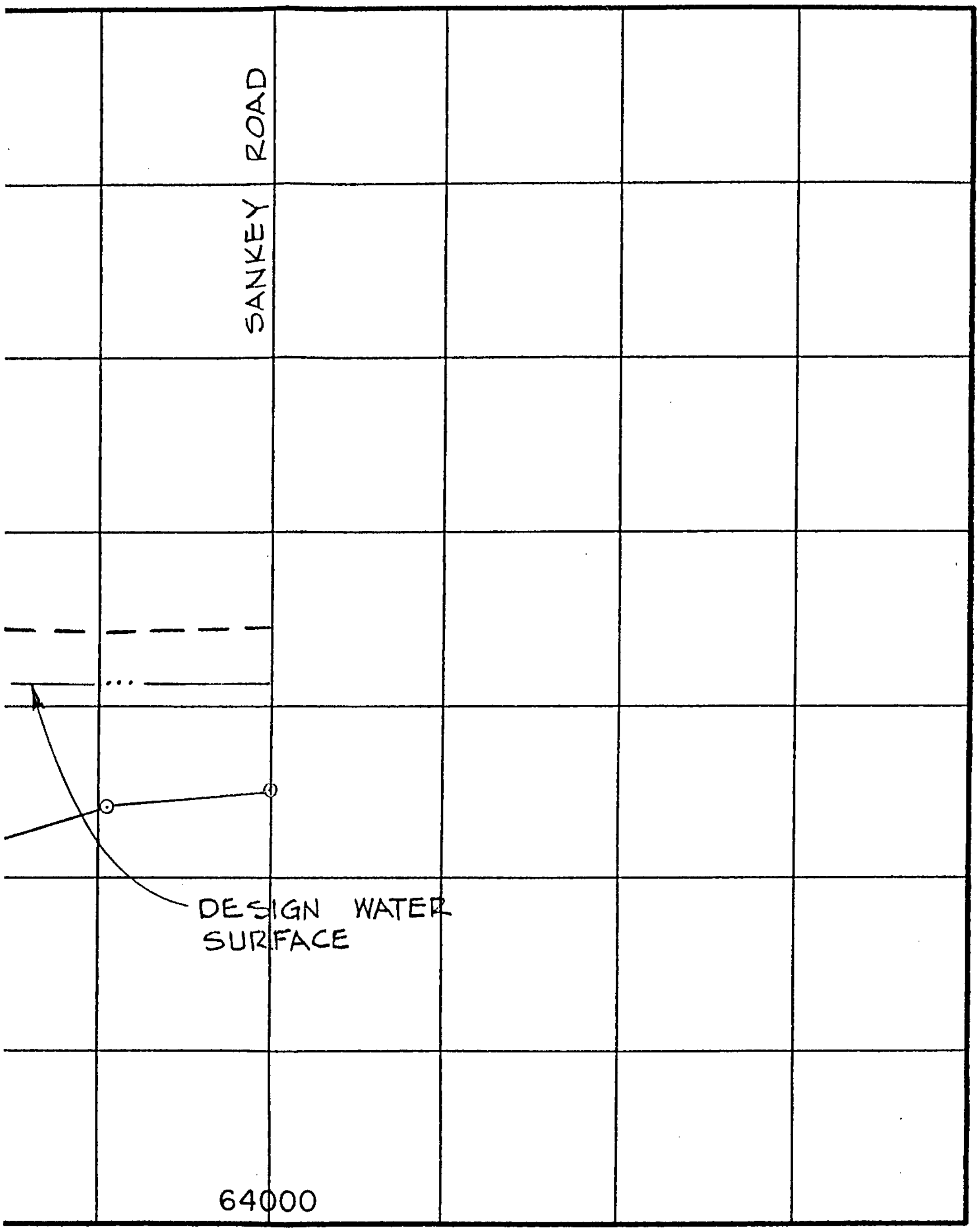
WATER

PLEASAN
WEST (LI
SEI

SACRAMENTO D

2



AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

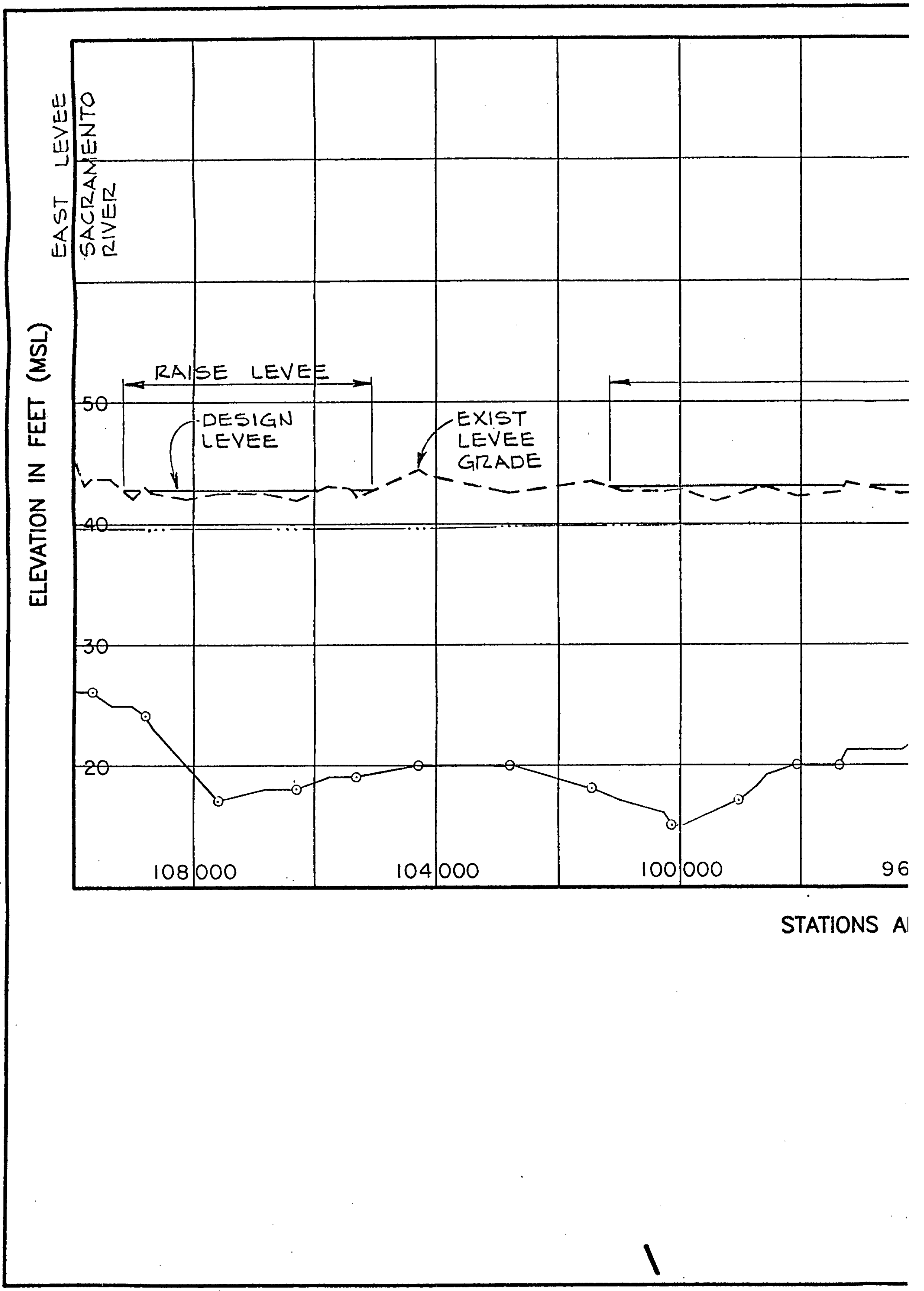
LEVEE CROWN AND
WATER SURFACE PROFILES

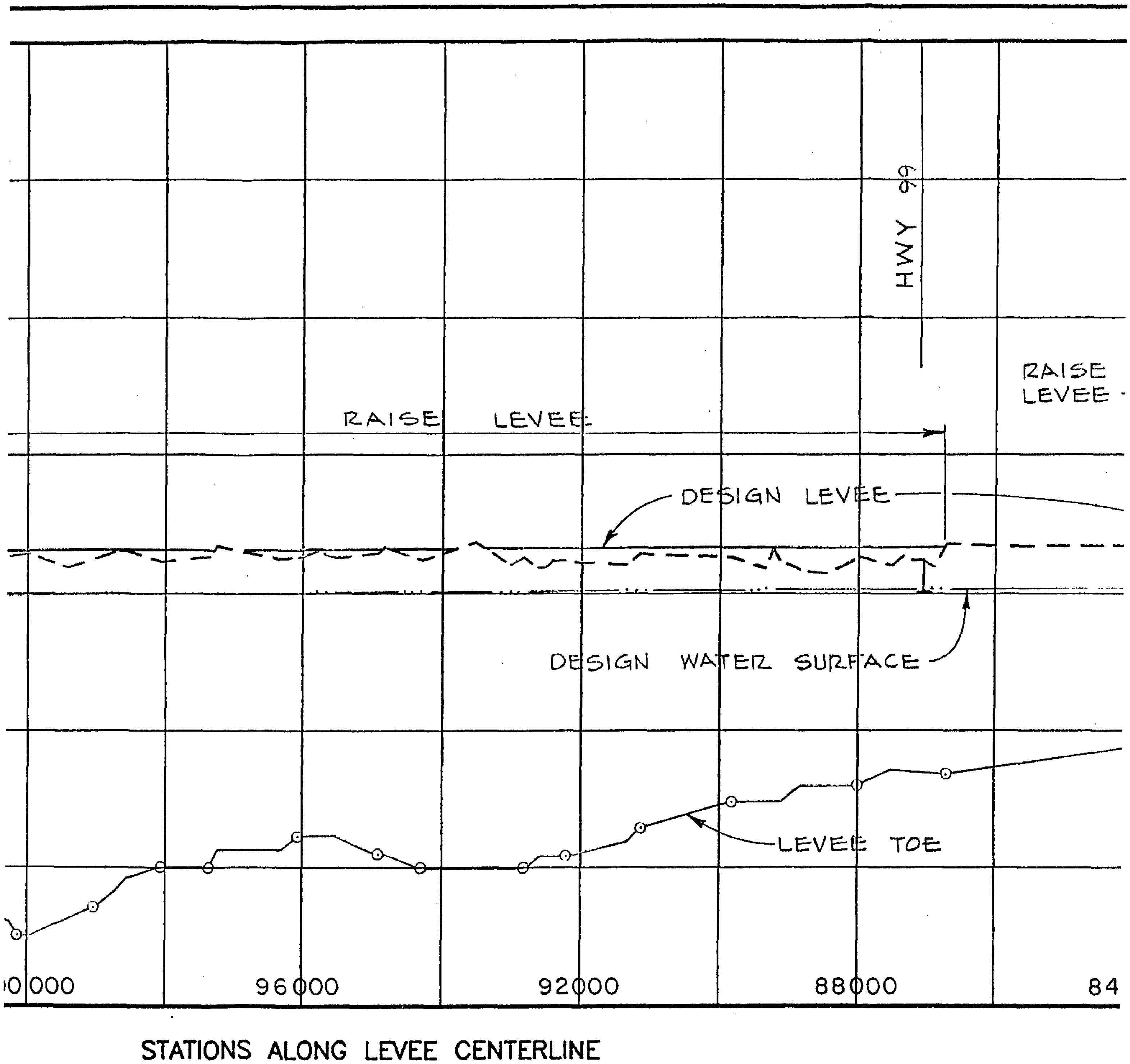
PLEASANT GROVE CANAL
WEST (LEFT) LEVEE PROFILE
SELECTED PLAN

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 10

3


EAST LEVEE SACRAMENTO RIVER
ELEVATION IN FEET (MSL)
RAISE LEVEE
DESIGN LEVEE
EXIST LEVEE GRADE
50
40
30
20
108 000
104 000
100 000
96
STATIONS A



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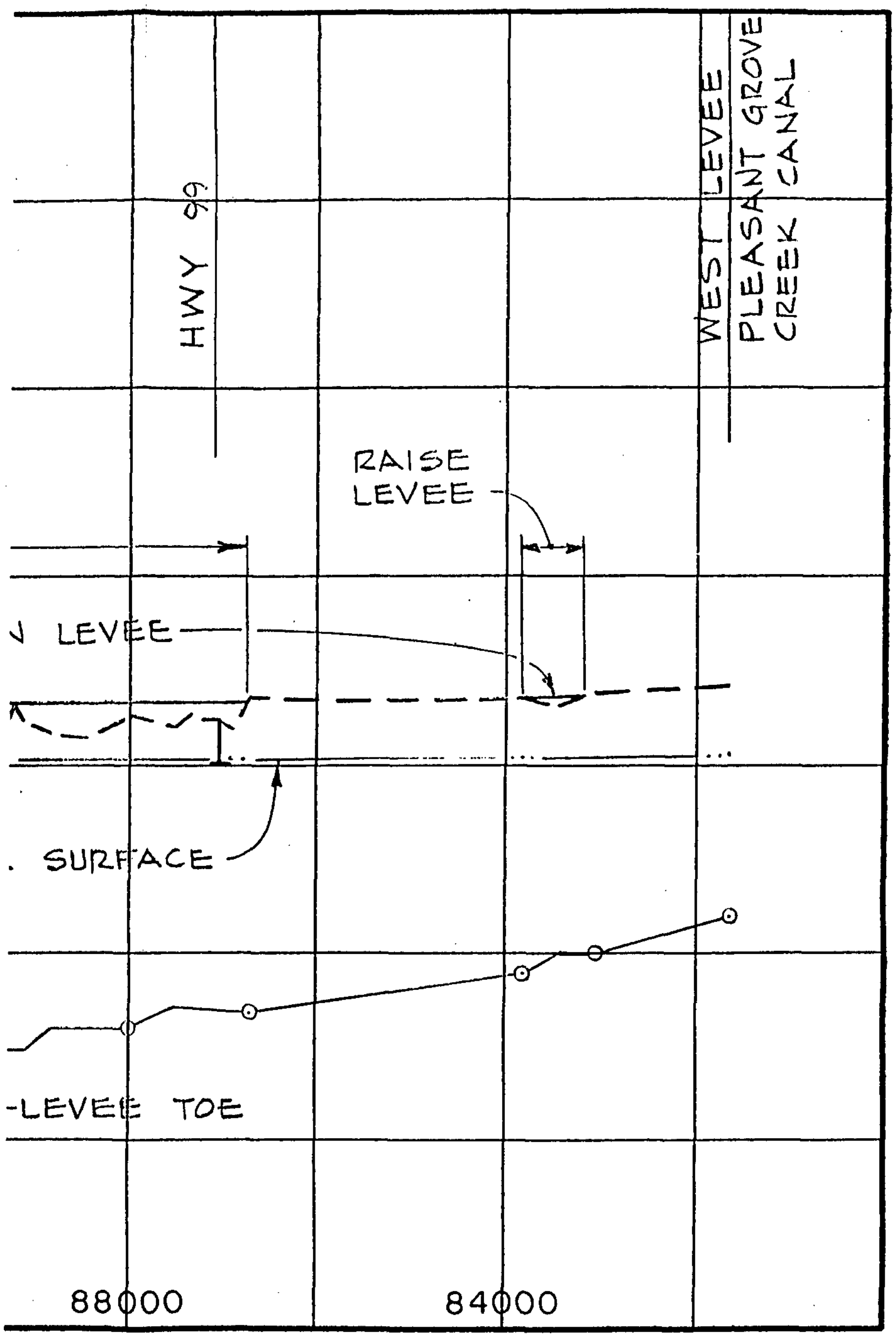

AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

LEVEE CROWN AND
WATER SURFACE PROFILES
NATOMAS CROSS CANAL
SOUTH (LEFT) LEVEE PROFILE
SELECTED PLAN

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 11

3

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

APPENDIX N

CHAPTER 2

BASIS OF DESIGN AND COST ESTIMATES

NATOMAS EAST MAIN DRAINAGE CANAL
PUMPING PLANT

A-E DESIGN

DECEMBER 1989

FEASIBILITY DESIGN
NATOMAS EAST MAIN DRAIN CANAL (NEMDC) PUMPING STATION
AMERICAN RIVER WATERSHED INVESTIGATION
FINAL SUBMITTAL

Contract No. DACW05-89-D-0023
Delivery Order No. 0002

Prepared For

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT CORPS OF ENGINEERS
CENTRAL VALLEY SECTION
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NATOMAS EAST MAIN DRAIN CANAL (NEMDC) PUMPING STATION
AMERICAN RIVER WATERSHED INVESTIGATION

TABLE OF CONTENTS

LIST OF TABLES AND FIGURES

NATOMAS EAST MAIN DRAIN CANAL (NEMDC) PUMPING STATION AMERICAN RIVER WATERSHED INVESTIGATION

SECTION I - PUMPING STATION DESIGN ALTERNATIVES

INTRODUCTION

The Feasibility Design Study for the Natomas East Main Drain Canal Pumping Station, American River Watershed Investigation, is authorized by the Flood Control Act of 1962 (Public Law 87-874).

This study is prepared for the Department of the Army, Sacramento District Corps of Engineers by Eckhoff, Watson and Preator Engineering, Salt Lake City, Utah. The purpose is to prepare a feasibility scope design, quantity take-off, cost estimate and written basis of design for a pumping station to be located on the Natomas East Main Drain Canal, approximately 1/2 mile South of Elverta Road and also approximately 7 miles upstream of the confluence of the Sacramento and American Rivers near downtown Sacramento, California. The design of levees upstream and downstream, and an embankment to the east of the pumping station are to be designed independently by the Corps and therefore are not included in this study. Additionally, the hydraulic and geometric design of a bypass channel to discharge low flows downstream is also to be designed by the Corps and also likewise is not included in this study. However, for purposes of overall project layout, the bypass channel has been shown on the plans and assumed dimensions are stated as such. See sheet 1 in Appendix D for an overview of the site plan.

The background need for the pumping station was initially referenced in the January 1988 Reconnaissance Report, American River Watershed Investigation, California, prepared by the Sacramento District, U.S. Army Corps of Engineers. The report outlined construction of a gated embankment structure and pumping station at the mouth of the NEMDC (approximately 7 miles south of the currently proposed site) and/or Natomas Cross Canal. During normal flow conditions, the gates on the embankment would be opened, allowing water from the canal to discharge downstream (through the bypass channel). During high flows in the American or Sacramento Rivers, the Gates would be closed, preventing river flows from entering the canal and causing backflow. Also, large capacity pumps in the canals at the structure would accommodate tributary inflows. The pumps would control the stages in the canal to avoid encroachment into the freeboard on the adjacent and upstream levees for specified design events.

For the NEMDC, a gated embankment structure at the mouth

would reduce the likelihood of upstream floodflow encroachment onto the canal levee freeboard and also help flow problems in the lower reach of Dry and Arcade Creeks. The Reconnaissance Report studies indicate that a pumping facility with a capacity of 12,000 and 15,000 cfs would be required to accommodate inflows primarily from Arcade and Dry Creeks from the 100- and 200- year events, respectively. Pump facilities of this size would be among the largest ever constructed. First costs for facilities to handle the two events would range from about $68 to $84 million, respectively. The costs of this measure would be significantly in excess of most appropriate substituted measures, i.e., levees along NEMDC. Accordingly, this measure was deleted from further consideration.

The current need for the pumping station is outlined in the 26 June 1989 Scope of Work for the Feasibility Design, Contract DACWO5-89-D-0023, Deliver Order No. 0002, a copy of which is included in Appendix E. The conceptual plan is to locate the pumping station approximately 7 miles upstream of the mouth of the NEMDC and approximately 1/2 mile south of Elverta Road. The plan is to reduce the risk of flood water overtopping the NEMDC levees upstream of the pumping station in a manner similar to that referenced in the Reconnaissance Report. However, since the pumping station is to be located upstream of Arcade and Dry Creeks it would not help flow problems in those areas. Instead levees downstream of the pumping station are to be raised to account for the higher stages resulting from river flows entering the canal and causing backflow and also resulting from pumping tributary flood inflows over the embankment structure. Pumping station alternative designs for 100, 400, and 700 cfs pump flows are to be studied to accommodate an inflow of 2,500 cfs from the 200-year flood event. Studies of costs for raising the upstream and downstream levees as conducted by the Corps along with the costs developed in this study for the pumping station will be utilized to determine which alternative pumping station size will be recommended for further design consideration. Construction is planned in approximately 1994-1996.

GENERAL

Three pumping station alternative designs were investigated for pump flows of 100, 400, and 700 cfs. The station designs were developed to the level of detail sufficient to permit making a cost comparison based on a first cost (October 1989 price level) that will not increase by more than 20 percent at the time of construction, excepting changes in the scope or function of the pumping station and/or adjustments due to construction cost index levels. The purpose of this study was to prepare the feasibility scope design, design criteria and cost estimates for the

alternatives. Other than adjusting the sizes of the pumping station components for alternative pump flows, no additional investigations were performed to optimize any design. The selected arrangement for further design consideration will be developed and optimized during the preparation of the Design Memorandum phase.

All alternative station flow capacities were designed using five pumps. Three large pumps each with pumping capacity equal to 30% of the station design flow and two smaller pumps, each with pumping capacity equal to 10% of station design flow, giving a total pumping capacity of 110% of design flow. The large pumps are diesel powered with the smaller pumps relying on electricity for power. The station is backed up with a diesel generator capable of supplying power for both the pumps and other electrical demands in case of electrical power outages. With outage of one small electrical pump and one large diesel pump or only one large diesel pump, the pumping station capacity would be 70% or 80%, respectively, of the design pump flows. This satisfies the guidelines outlined in EM 1110-2-3102, paragraph 5. "Selection of Size and Number of Pumping Units" that with consideration that one pumping unit may become inoperative, the number of units provided should be such that two-thirds of the station capacity should be available.

In general, the pumping station works in principle as an embankment structure located across the canal to prevent high flows in the American River from causing backwater in excess of the levee heights upstream of the pumping station. During these high river flows, the gates in the bypass channel (as being designed independently by the Corps) would be automatically closed and the pumping station would pump any tributary inflows over the embankment structure. During normal flow conditions (flow elevations lower than elevation 28.0) the bypass channel gates would be open, allowing tributary inflows to pass downstream. The required heights for the raising of the existing levees both upstream and downstream of the pumping station have been determined by the Corps in their studies and are not addressed in this study.

Project operation and maintenance will be the responsibility of an as yet unspecified organization. However, the City of Sacramento, Flood Control and Sewers Division has been identified as a potential operator and thus has been contacted during the feasibility design to determine their current preferences for fuel sources, operations and maintenance procedures and practices, construction materials, and station layouts. The pumping station is anticipated to be set for unattended operation with automatic controls for the bypass channel gate operation and pump sequencing. Remote alarms will be installed to alert the operating organization in case of an emergency. The maintenance and operation of the pumping station is further described in Section II. Operating and Maintenance Procedures.

SITE CONDITIONS

Accessibility to the pumping station for operation and maintenance can be provided from the East Levee Road located immediately west and adjacent to the NEMDC. See Sheet 1 in appendix D. The East Levee Road is proposed to be raised by the Corps as part of the downstream levee raising plan, to elevation 43.5 which is beyond the scope of this study. Therefore, the grading of the access road will need to be revised when that raising takes place. Also accessibility to the pumping station in case of an emergency can be obtained from the proposed downstream levee adjacent and west of the railroad on the east side of the NEMDC. That levee is also being designed by the Corps and is beyond the scope of this report, but its approximate location is shown for reference. The west downstream levee can be accessed through the town of Rio Linda.

Parking for maintenance vehicles is provided on the west side of the pumping station in a graveled surface area. This area also provides a minimum 45 foot radius turnaround for maintenance vehicles and fuel delivery trucks. The entire pumping station and embankment structure including access ramps and parking areas are fenced and lighted for security and night time operations.

The location of the pumping station has been moved approximately 100 feet to the north of the original location identified with the Corps during the site visit. This was required to minimize interference of the bypass channel with a near by power transmission tower approximately 180 feet downstream of the station's afterbay.

The topography of the site required that the existing NEMDC channel be widened to facilitate the pumping station and bypass channel width. The transition from the existing channel slopes to the pumping station were gradually made at a 4 longitudinal to 1 transverse angle to minimize head loss, turbulence, and erosion of the new channel side slopes. The new channel and side slopes will have rockfill slope protection to elevation 28 and grass seeding above that to assisting in minimizing erosion. Reference to geothechnical investigations and recommendations is directed to Section III of this report.

The pumping station is located in the area of the existing channel and right bank area. The bypass channel is located in the left bank area. To facilitate the channeling of low flows to discharge through the bypass channel, the incorporation of a proposed diversion dike or wall, similar to that shown on Sheet 1, is to be included by the Corps in the design of the bypass channel and levees. This diversion dike will also assist in diverting flows for care of water during construction.

The tightness of the site restrained the location of the fuel storage tanks. The only area that has a minimum 150 foot

clearance from all property lines as required by fire protection code regulations is near the northeast corner of the pumping station deck. A power transformer pad was therefore located on the west side of the pumping station.

The canal bottom slopes and side slopes will be protected from erosion by rip-rap and either a geotextile fabric or graded soil filter layer. For estimating purposes a 12 inch thick rip-rap layer was placed in the area 20 feet upstream of the forebay deck and a 48 inch thick layer changing to a 12 inch layer in 20 feet was placed in the area 40 feet downstream of the afterbay deck for all pump flow alternatives. Further geotechnical investigation and model flow studies should be conducted at the final design stage to select the final rip-rap and other hydraulic requirements.

HYDROLOGY AND HYDRAULICS

The scope of work required that the hydrology and hydraulics, as designed by the Corps, be addressed in terms of the flows to be accommodated by the pumping station and the stages to be maintained in the NEMDC. Stage curves for 500, 700 and 1200 cfs pumping stations for 100, 200 and 100 year events, respectively, were made available during the negotiation of the scope of work for the project, but when the pump flows were deduced to the current lower capacities, no stage curves were available. To adjust for the lack of stage curves to relate upstream and downstream water surface elevation's for each pump flow alternative, the Corps stipulated minimum and maximum water surface pool operating levels of 30.0 and 39.0, respectively. These levels could occur on either the upstream or downstream sides of the pumping station during operation in a flood event. These water surface elevations were to be used for all three alternative design flows.

The downstream levees were proposed to be raised to an elevation of 43.5 (39.0 + 3.5 feet of freeboard). Currently the elevation of the levee in the vicinity of the pumping station is 39.6 as indicated in the GR card printout of the Corps HEC-2 studies (between x-Sec. No. 9.672 and 9.759). From this information, the engine deck of the pluming station was instructed by the Corps to be at elevation 44.0.

The tributary inflow from a 200 year flood event was specified at 2500 cfs. This flow was used to determine preliminary sizing of the bypass channel and also the estimated sizing of pipes to handle the care and diversion of water during construction.

The bypass channel should be capable to pass low and normal flows and also to some extent higher flows during an emergency. The field reconnaissance suggested that the low and normal flows were contained within the existing low flow channel. From the 1" = 2000' scale topography maps and the x-sectional information provided by the Corps' HEC-2 printouts,

the top of bank elevation of 28.0, channel invert elevation of 23.4 and average width of 35 feet were identified. Based upon an estimate flow velocity of 2 fps, the normal channel capacity was estimated as approximately 300 cfs (35' x 4.6' x 2 fps). Therefore, using a slightly higher flow velocity of 2.5 fps for flows discharging through the bypass channel, a preliminary width of 25 feet was selected (300 cfs ÷ 4.6' x 2.5 fps). This preliminary sizing of the bypass channel as stated previously was performed only to assist in overall project layout since the bypass channel is an integral part in the consideration of locating the pumping station.

During higher flows in the NEMDC, the functioning of the bypass channel to accommodate those flows prior to the closing of the gates is beyond the scope of this report. The current planning, as outlined in this report, for the initiation of the pumping station is to close the bypass channel gate and start sequencing of the pumps at a water surface pool elevation of 28.0, with all pumps operating at an upstream pool elevation of 30.0. During backwater flows into the NEMDC from the American River, the closing of the gates and pumping operations described above are expected to yield desired results. However, if tributary inflows and not river backwater flows are the cause of higher water surface elevations immediately downstream of the pumping station, then the closing of the bypass channel gates and initiation of pumping may be an unnecessary action as opposed to keeping the bypass channel gates open to discharge the tributary inflows downstream. The resolution of these considerations are beyond the scope of this report and should be addressed by the Corps in their design of the bypass channel.

COST COMPARISON

Cost estimates for this feasibility design stage were prepared under guidance outlined in EM 1110-2-1301, Engineering and Design Cost Estimates - Planning and Design Stages; EM 1110-2-538, Civil Works Project Cost Estimating - Code of Accounts; EC 1110-2-263, Civil Works Construction Cost Estimating; and EDM No. 46, Basis of Cost Estimates for Civil Works.

The estimates of cost made during this feasibility design have attempted to embrace the entire project and include the cost of all work necessary for a complete job, ready for operation. Except for the costs required for the bypass channel, diversion dike and the embankment tying into the west downstream side levee (which are to be designed by the Corps as identified on the drawings), all the ultimate requirements of the project have been attempted to be visualized and allowance for all items included. In this feasibility design stage, it has been necessary to allow for quantities of materials not yet definitely determined by detailed design and to include larger contingencies for items not yet known

because of lack of detailed investigations. The degree of refinement at this stage of design is therefore at a level consistent with the engineering guidance manuals referenced above.

Cost estimates have been based upon current bid prices or historical cost indexes such as the Means Construction Cost Estimating Guides. The unit cost of items have also been adjusted as necessary to fit the October 1989 price level and the general region in which the project is located. Where cost data on work of a similar nature was not available, the costs were arrived at by estimating equipment, materials and labor plus allowance for contractors overhead and profit. It was also speculated that the general contractor would likely handle the civil and heavy construction items of work and subcontract the mechanical and electrical items.

At this stage of the project development, the allowance for contingencies against some adverse or unanticipated condition not susceptible to exact evaluation from the data at hand or uncertainties beyond the control of the estimator has been represented as a percentage factor based upon the judgement of the estimator and following suggestions in the guidance manuals. The contingencies item is not an allowance for omissions of work items which are known to be required, but rather it is an allowance for the estimated level of quantities which may have not yet been determined by specific or detailed design stages of project development.

Costs for engineering and design, and supervision and administration (inspection) are also presented on the feasibility cost estimate. These costs are also presented as a percentage factor based upon judgement and reference to prior Corps project cost estimates.

The breakdown of the estimate into features, subfeatures and subfeature elements using the cost estimate check lists and standard code of accounts of EC 1110-2-538 has been followed with some slight modifications. Account numbers for subfeature elements of permanent access roads and parking, canals, and associated general items have been added to the Pumping Plant feature code and cost estimate checklist.

The cost estimate is for a mixed large diesel and small electric driven pumping station as described in Section II. An analysis of the costs associated with an all diesel driven and an all electric driven pumping station alternative was performed to compare costs with the use of the mixed drive configuration. For the 400 and 700 cfs alternatives, the all diesel driven pumping station construction cost was $359,300 and $348,900 less than the mixed driven arrangement, respectively. The 100 cfs alternative for all diesel driven was not compared. For the all electric driven pumping station, the cost increase over the mixed driven arrangement was not as consistent. The construction cost for the 700 cfs, all electric alternative was $73,500 more than the mixed driven arrangement. The 400 cfs alternative was approximately

the same price. The 100 cfs alternative for all electric driven pumps was $193,300 more than the mixed driven arrangement. Refer to Appendix C for cost estimate sheets that compare the drive alternatives. The decision to use the mixed drive configuration for design is outlined in further detail in Section II.

The cost estimates for the three alternative flow pumping stations are summarized in Table N-2-1 by subfeature codes. Refer to Appendix C for detailed quantity take-offs and cost estimates of each subfeature element.

RECOMMENDATIONS

The basis of design and cost estimates developed in this study for each pumping station alternative will be included in a Documentation Report that will be prepared by the Sacramento District, Corps of Engineers. The Documentation Report will contain design and technical data that will support a recommended plan of improvement for the raising of levees and construction of both the bypass channel and pumping station. That plan will be described in a Feasibility Report, also to be prepared by the Sacramento District.

TABLE N-2-1
COST ESTIMATE SUMMARY FOR
ALTERNATIVE NEMDC PUMPING STATION

ACCOUNT NUMBER	DESCRIPTION OF ITEMS	100 cfs	400 cfs	700 cfs
13.0.-.-	PUMPING STATION			
.0.A.-	Mobilization, Demobilization & Preparatory Work	127,000	200,500	250,000
.0.B.-	Care and Diversion of Water	57,600	57,600	57,600
.0.C.-	Permanent Access Roads & Parking	159,750	167,700	178,600
.0.D.-	Excavation for Structures	16,570	24,642	32,400
.0.1.-	Pumping Station, Substructure	1,024,600	1,304,275	1,500,225
.0.2.-	Pumping Station, Superstructure	9,360	9,360	9,360
.0.3.-	Pumping Machinery & Appurtenances	233,000	626,200	925,000
.0.4.-	Gates and Valves	90,700	168,700	281,900
.0.5.-	Auxiliary Equipment	177,700	449,200	468,500
.0.6.-	Utilities	500	500	500
.0.7.-	Canals	24,525	34,775	41,500
.0.R.-	Associated General Items	20,775	21,275	21,575
.0.Z.-	Contingencies	485,550	770,000	942,000
ENGINEERING & DESIGN		291,350	460,200	565,100
SUPERVISION & ADMINISTRATION		194,200	306,800	376,750
	TOTALS	2,913,280	4,601,727	5,651,010
ANNUAL O & M COST @ 1% of first cost + $55/HP[1]		40,777	93,347	157,092
ANNUAL REPLACEMENT COST @ 60% of first cost[1], 100 year design life, 8 7/8% interest (Total x 60% x 0.31%)		4,516	7,133	8,759

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[1]. EDM No. 46, Basis of Cost Estimates for Civil Works, 1985, SPIKED-T. HP is 300, 1000, and 2000, respectively.

SECTION II - PUMPING STATION

PUMPING STATION LAYOUT

General

The site plan, pumping station plan and longitudinal section are shown on Sheets 1 through 3 of Appendix D. The pumping station for the 700 cfs pump flow alternative is shown to scale and detailed with dimensions and criteria for the other pump flow alternatives shown in tables. There are four pump bays leading to five pumps. A single bay feeds water to the two small electrical driven pumps, with the remaining three bays supplying water to three diesel engine driven pumps.

A fifth bay is added to the east side of the plant for a bypass channel which allows "nuisance" flows or low flows to discharge downstream of the pumping station. There is a low diversion dike or wall which will direct low flow waters to this fifth bay, therein preventing silt and trash from accumulating in the forebay area. The design of this bypass channel and diversion dike is not included in this study.

Access

As described earlier, primary access to the pumping station will be from the East Levee Road via a turnout. The East Levee Road is proposed to be raised to elevation 43.5 which will approximately match the deck elevation of the station. The East Levee Road will also be the connection point for telephone and power to the station. Existing aerial phone cables run along the west side of the road, and it is anticipated the power will be run along the east side of the road.

The forebay deck, capable of handling maintenance trucks and construction equipment, will provide access across the station to the east side. The forebay deck will also be used to collect trash removed from the bar screens by the catenary trash rakes. This trash can be loaded and hauled away. The diesel fuel tanks and a parking area will be located on the east side of the plant. This parking area will have a minimum 45 foot turning radius for large vehicles.

The Corps of Engineers will design a proposed levee that will extend eastward from the parking area embankment. This will serve to contain downstream backwaters.

Deck Layout

The pumping station deck is set at elevation 44.0, this will provide 5 feet of freeboard for the flood scenario. The pumps will be suspended from the station deck with discharge to the south below the deck. For the large diesel pumps, the

gear reducers and diesel engines will be mounted on the deck. For the small electric pumps, the motors will be mounted on the pumps. A small, 300 gal diesel day tank will provide fuel to the diesel engines should the large tanks require maintenance or filling. Access hatches with fixed ladders will allow entrance through the deck into sumps to allow visual inspection and also allow for dewatering pumps to be lowered. Dewatering pumps will be supplied by the operating organization. Manual handwheels which raise and lower the sluice gates are mounted on the deck. Removable grating in the afterbay deck over the discharge flap gates will allow for accessibility for maintenance purposes. A small building located on the east side of the deck will contain the electrical panels and motor controls, standby diesel generator and an operations office.

Future Considerations

The engine deck has been sized to allow future addition of a building. Refer to sheet 2. This building would enclose the mechanical equipment to reduce noise levels should the residential development occur in the area and near to the site. Extra deck space has been allowed for silencers and mufflers inside the building to aid in the reduction of noise.

MECHANICAL SYSTEM DESIGN CONSIDERATIONS

General

The following section discusses factors common to the mechanical system design considerations for all three pumping station sizes. Subsequent sections discuss design considerations specific to each size pumping station. Specific equipment manufacturers and models are identified below in discussions of each station, and are described in the catalog data sheets in Appendix A. Calculations are provided in Appendix B.

Flood Water Elevations. - The Sacramento District Corps of Engineers, Central Valley Section, specified maximum water elevation of 39 feet (above mean sea level) downstream and 30 feet upstream of the pumping station. The first pump shall be started when up-stream water elevation is 28 feet. Pump starting and sequencing shall be controlled in uniform increments throughout the two foot range between 28 and 30 feet. Pumping capacity equal to 100 percent of specified station design flow shall be provided at 30 foot upstream water elevations.

Pump bay dimensions were determined from Plate No. 22 in

Manual No. 1110-2-3105[2] to the maximum extent possible. Dimensions not shown in Plate No. 22 were estimated using the Hydraulic Institute Standards,[3] either directly from Figure 81, or with modifications. (Copies of the tables and figures used are provided in Appendix B).

Pump bay floor elevations for the 700 and 400 cfs stations were determined from Figure 81 and raised by about 40 percent to reduce bay construction costs. Minimum bay water depths are 26 percent or greater than the minimum water depths recommended by 1110-2-3105 for pumps with suction lift capabilities.

Pump Capacities. - Five pumps will be used. Three large pumps, each with pumping capacity equal to 30% of station design flow and two smaller pumps, each with pumping capacity equal to 10% of station design flow. With all five pumps on line, pumping capacity will be 110% of specified design flow. At the assumed worst case pump outage of one small and one large pump inoperative, pumping capacity is 70% of design flow. This is slightly more capacity than the 2/3 (67%) capacity at maximum outage recommended by EM 1110-2-3102. See calculations in Appendix B.

Pump Selection. - Axial flow (propeller) type pumps will be used throughout, such as manufactured by Cascade Pump Co. or Johnston Pump Co. These pumps show good efficiencies (76% to 86%) at the design conditions and good suction lift capability at near zero submergence. Umbrellas are recommended at the pumps suction entrances. See calculations in Appendix B. Pumps will be under floor discharge to allow mounting the speed reducers and drives on the main floor of the station. Also, maintenance access is more convenient and safer when maintenance personnel do not have to work on elevated platforms.

Pump Drive Selection. - Discussions with the City of Sacramento, Flood Control and Sewer Division personnel indicate that electric motor drives are first choice, second choice is diesel engines, and third choice is propane fueled engines.

Of the many pumping stations operated by the City of Sacramento, only one (San Juan) uses diesel engines as the sole power source. Air pollution concerns appear to make it unlikely that in the future all diesel powered stations will be constructed in the Sacramento area.

Three pump alternatives were considered: all electric powered, mixed diesel electric, and all diesel. The first alternative includes 100 percent electric power backup by

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[2]Manual No. 1110-2-3105, "Engineering and Design, Mechanical and Electrical Pumping Stations", 10 Dec. 62.

[3]"Hydraulic Institute Standards for Centrifugal, Rotary and Reciprocating Pumps", Fourteenth Edition, 1983.

diesel-electric generators. The second alternative uses electric motors to drive the two smaller pumps (most frequently operated) and diesel engines to drive the three larger pumps (operated during emergency, flood conditions). This second alternative provides a compromise between first costs and pollution abatement. Diesel-electric generators would provide backup power for the two small electric pumps, and other station electrical loads.

Cost comparisons show that for the 100 and 700 cfs stations, the all electric drive stations have highest first costs, followed by the mixed electric and diesel, with the all diesel drive stations the lowest first costs. For the 400 cfs station, the mixed drive station has the highest cost, followed by the all electric drive station, with the all diesel drive station the lowest first cost. The designs described herein are for mixed drive pump stations; i.e., two smaller pumps with electric motor drives and three larger pumps with diesel engine drives.

The diesel engines will be two or four stroke, turbocharged or naturally aspirated (depending upon horsepower requirements), water cooled (with air cooled radiators), six cylinder (in line), with 24 volt auto-electric starting. Engine speeds will be governor controlled. Safety shutdown sensors and devices will be provided. Dry exhaust manifolds with silencers will be provided. Remotely controlled clutches (air or electric actuated) would be provided.

The selections are conservative in that smaller engines, operating at higher speeds and/or BMEPs[4] could be used. Engine rotational speeds and BMEPs are below the upper limits provided in Tables B and C of Section 16263[5] for engine operation Class C[6]. The Waukesha engines referenced are heavy duty, have longer intervals between overhauls, and higher first cost than some other brands. See discussion at Operating and Maintenance Procedures.

Electric motors will be vertical, hollow shaft, integral thrust bearings, mounted on top of the pumps. Motor voltages will be 480 volt for the 100 cfs station and 4160 volt for the 400 and 700 cfs stations. See additional discussion in the Electrical System Design Considerations section.

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[4]BMEP - Brake Mean Effective Pressure

[5]Corps of Engineers; Specification Section 16263; "Diesel-Generator Set, Stationary, 100-2500 KW, with Auxiliaries", April 1985.

[6]Class C: 1 to 5 day continuous operation at 100 percent of rated load; less than 1,000 hours/year; less than 10,000 hours/10 years.

Speed Reducers - Right angle, floor mounted, speed reducers will be used at all the diesel driven pumps. See catalog data sheets for Amarillo Gear Co.

Fuel Storage and Supply System - The fuel storage and supply system described herein was sized to accommodate an all diesel drive, 700 cfs capacity pumping station. All electric, or mixed diesel and electric drive 700 cfs pumping stations require similar fuel storage capacities since 100% backup diesel-electric generating capacity is provided. In view of the small percentage of the total station costs attributable to the fuel storage and supply system, no cost adjustments are made for the smaller stations. The guidance/assumptions used as basis for preliminary design of the fuel system are as follows:

1. Pumps will be diesel-powered (#2 fuel oil).
2. Standby generator will be diesel-powered.
3. Above-grade tank storage is preferred.
4. Tanks will be non-pressurized (atmospheric).
5. Tanks will be of steel construction.

(1) Storage Capacity. Fuel volume required on-site was calculated based on the fuel draw of 100 gal/hr (for maximum 2000 HP loading at 700 cfs station design capacity) by the pumps, plus the generator draw of 12 gal/hr. Fuel need for the pumps was based on a 30 hour run and for the generator, on an 8 hour run, resulting in a total of about 3400 gallons. The storage gallonage was assumed split between two tanks, a 10% contingency was added, and the volume rounded up to the next readily available tank size, resulting in the recommendation to use 2 each, 2500 gallon, above grade tanks.

(2) Construction. The construction of the tanks shall be in accordance with UL 142-1972, as dictated by paragraph $5585 of California OSHA Title 8 regulation, Article #145, "Tank Storage".

(3) Layout. The general arrangement of the tanks was made to be in compliance with paragraph $5589 of the above cited Cal-OSHA regulation.

The #2 fuel oil is a Class II flammable liquid, and paragraph $5589 provided the applicable guidance for locating the tanks. Both tanks must be at least 150 feet from the nearest property line (including the opposite side of a public way), and have at least 3 feet clear between tanks. These rules also dictate on-site spacing to any "important building"; however, there are (and will be) no on-site building which

meets the regulatory definition of "important building".

(4) Diking. The two storage tanks are mounted on above-grade saddles, and the area is bounded by an 18" high concrete perimeter containment dike, as shown. The area also has a concrete slab floor, with drain sump. The liquid containment capacity within the diked area is equivalent to the entire volume of 1 tank, plus precipitation generated by the 25 yr - 24 hr rainfall event, plus a freeboard and safety allowance. No provisions have been included for removal of liquids (either rainwater or leaked fuel) from within the containment dike. A pump (either dedicated or portable) might be required for this function or, if elevations permit, a valved drain line may be used to gravity drain these liquids. In either case, a non-automatic draining operation is indicated, since a characterization of the liquid (fuel or water) would be required before the liquid could be removed and released to the environment,

(5) Miscellaneous. Ancillary equipment considered and included, within the Fuel Storage and Supply System are:
1. A 300 gallon, elevated day tank located near the diesel engines.
2. Connecting piping between the storage tanks and the day tank.
3. Fuel pump (storage to day tank).
4. Fuel filter.
5. Level sensing/control for day tank.
6. Handling equipment and access equipment (platforms, ladders, etc.) for fuel tanker unloading.

Forebay Gates - The sumps of each pump will be equipped with rectangular sluice gates on the inlets to isolate the sumps from the upstream ditch. This will allow the sumps to be pumped dry for maintenance purposes, and minimize silt accumulation. The discussion below is for the 700 cfs station; similar parameters and calculations apply to the other stations.

All gates will be slide type, sized as shown on the drawings (nominal 60" x 72" size for the 700 cfs sta.) with vertical rising stems equipped with crank-type manual operators. The 700 cfs station will have 2 gates per sump, the other stations are as shown on the drawings.

Gate openings were sized to maintain a water velocity of under 5 fps when flowing full. The flow velocity based on

opening area (see calculations) for the large pumps is about 3 1/2 fps, and for the small pumps about 2 1/2 fps.

Trash Control - The discussion below is for the 700 cfs station; similar parameters and calculations apply to the other stations. The inlet flow to each pump will be screened by trash racks in front of the forebay gates. The rack bottom will be at an elevation of 21 ft. and the width will span each forebay. Bay openings will be 16 ft. (clear), with 4 ft (min.) required between bays to provide space for rake mountings/drives. This results in overall minimum module widths of 20 ft. Racks are inclined approximately 60° from the horizontal.

The racks will be conventional bar screens with 3/8" x 3" steel bars, mounted vertically, with 2" clear spaces between bars. With a water elevation of 28 ft (start pump elevations), flow velocity through the large pump racks will be 2.2 fps, and through the small pump rack 1.5 fps.

Racking of all trash racks will be accomplished by use of mechanical, catenary-type rakes. These units (4 total) will be equivalent to those currently in use at several Sacramento pump stations as manufactured by the E&I Corp., Westerville, Ohio.

Each rake will be driven by an electric motor (3-5 HP) and will be arranged such that the raked trash will be deposited upon a deck near the top rear of the unit. This deck will be of sufficient width and strength to allow working room for a front end loader and for passage of trucks across the structure (from bank to bank).

Flap Gates - Flap gates, sized as shown on the drawings, will be used at the exits of the pump discharge piping. They will prevent backflow through non-operating pumps and preclude children who may intrude onto the site from entering the pump discharges. The flap gates are fully submerged at 39 foot downstream water elevation. Bronze seat faces in gate frames seal against neoprene seats in the covers. Gate bodies and covers are cast iron. A leaf spring bumper with rubber cushion is provided on the gates to prevent damage to the covers when the pumps start. See catalog data sheets for Waterman, Model F-55 Drainage Gates in Appendix A.

Dehumidification - No dehumidification equipment is planned for this installation. Electrical equipment, motors, and enclosures will be protected from the effects of humidity by provision of local electric heaters where required. The cost estimates reflect the inclusion of these heaters.

Operating and Maintenance Procedures - The pumping station controls will be designed for automatic, unattended operation. When water elevation reaches 28 feet (above mean sea level) automatic or remotely controlled sluice gates in the Corps designed bypass channel will close. One of the small pumps would start pumping. If upstream water elevation increases approximately 0.2 foot, the second small pump would start. Another 0.2 foot rise, and one of the large pumps

would start and the two small pumps would stop. The cycle repeats until all the large pumps and one small pump are operating (equals 100% of station design capacity). The second small pump could be started if desired.

The City of Sacramento performs routine maintenance and inspection of their pumping stations monthly.

Diesel engines are started and run for about one half hour per month. Electric motor driven pumps are not normally operated so as to avoid electrical peak demand charges.

Equipment inspection and maintenance are performed in accordance with written procedures. These procedures are based up on equipment manufacturers' recommended practices, modified as required by local maintenance experience and/or practice.

The City practice is to reset the pumping station controls periodically so that pump usage is rotated between the pumps (Lead-Lag system). This distributes wear approximately evenly among the pumps.

Waukesha brand engines are referenced herein and used to estimate engine costs for the larger pumps. These are heavy duty engines and well liked by the City of Sacramento. Less expensive engines are available, however, they may have higher maintenance costs and shorter intervals between major overhauls. For example, Waukesha engines of the sizes used herein operate 20,000 to 30,000 hours between major overhauls and maintenance costs are estimated at 0.0033 to 0.0042 $ per hp-hr. Similar horsepower Scania engines operate 12,000 to 15,000 hours between major overhauls and maintenance costs are estimated at 0.008 $ per hp-hr. Scania engines first costs are roughly 45% of Waukesha first costs.

No approved hydrographs have been provided for the various stations, thus engine operating hours during the 100 year station life have not been estimated. Therefore, no economic analysis of engine selection has been attempted.

Mechanical Design - 100 CFS Pumping Station

<u>Pump Sizes</u> - Three 24" (30 cfs), 700 RPM, 65 bowl HP and two 14" (10 cfs), 1170 RPM, 22 bowl HP, axial flow pumps will be used. Pump efficiencies are 70% and 76% respectively. See calculations in Appendix B.

<u>Pump Drives</u> - Three diesel engines, operating at 1750 RPM, will drive the large pumps through right angle speed reducers. Engines would be similar to Lister Petter (Hawker-Siddeley) model CS6. (See catalog data sheets.) Two vertical shaft, 20 HP, 1200 RPM, 480 volt electric motors, directly mounted on the pumps, will drive the small pumps.

Mechanical Design - 400 CFS Pumping Station

Pump Sizes - Three 42" (120 cfs), 440 RPM, 245 bowl HP, and two 24" (40 cfs), 700 RPM, 100 bowl HP, axial flow pumps will be used. Pump efficiencies are about 85% and 80% respectively.

Pump Drives - Three turbocharged diesel engines, operating at about 1300 RPM, similar to Waukesha model F18DS, operating at about 145 BMEP, will drive the large pumps. Two vertical shaft, 100 HP, 720 RPM, 4160 volt electric motors, directly mounted on the pumps, will drive the small pumps.

Mechanical Design - 700 CFS Pumping Station

Pump Sizes - Three 54" (210 cfs), 350 RPM, 480 bowl HP, and two 36" (70 cfs), 435 RPM, 160 bowl HP, axial flow pumps will be used. Pump efficiencies are about 86% and 78% respectively.

Pump Drives - Three turbocharged diesel engines, operating at about 1600 RPM, similar to Waukesha model H24DSI, operating at about 180 BMEP, will drive the three large pumps. Two vertical shaft, 200 HP, 450 RPM, 4160 volt electric motors, directly mounted on the pumps, will power the two small pumps.

ELECTRICAL SYSTEM DESIGN CONSIDERATIONS

General

The following section discusses factors common to all three pumping station sizes. Subsequent sections discuss design considerations specific to each size pumping station.

The electrical systems design will provide simple, compact, and reliable operation. Since the equipment will be subject to long periods of inactivity, level and simplicity of standby maintenance has been a prime consideration. Equipment designs incorporate provisions for prevention of corrosion and condensation effects. Also, since the equipment will be primarily unattended, equipment protection from vandalism and other damage should be provided, such as limited access, vandal proof external fixtures, and protection from dust and water contamination.

Electrical Utilities - Incoming power to the utility company connection point (assumed to be at the secondary terminals of the pad mounted transformer) will be an overhead line at a voltage level of 12,470 volts as recommended by the supplying utility company. In order to provide the most reliable service, the utility company (SMUD)[7] proposes to tie

[7]SMUD - Sacramento Municipal Utility District

to existing feeder lines along Elverta Road north of the station and along Elkhorn Road south of the pumping station, with a new line running along East Levee Road to form a loop. Cost estimates for utility company provided equipment are given in the discussions for each station size. These estimates include costs for transmission lines, distribution lines, and utility provided transformers and feeder line switching equipment. Since this equipment would normally be furnished at no cost to a government owner, power line costs are not included in the cost estimates for the pumping station, and are listed separately for reference purposes only. If the pumping station were to be privately owned, the owner would bear these power system costs. (SMUD has no responsibility or obligation for these estimated costs. An accurate estimate can be provided only after detailed plans are available to SMUD).

Transformers have been be sized to the closest standard size which will handle the normal equipment operating load. The design load for each station size includes adequate power for electric pumps, trash rakes, lighting, security, electric HVAC if used, and controls.

The pumping station operating voltages have been selected to limit surge currents (motor inrush currents) required for starting large motors while allowing across-the-line starting. The voltages selected are described in the discussion for each pumping station size. If voltages greater than 480 volts are required to allow across-the-line starting of electric pump motors, a separate, higher voltage buss will be provided for connection of the pump motors.

The transformer pad for installation of the utility company provided transformer has been sized at 20 feet square to accommodate the transformer required for the largest pumping station. In accordance with the NEC[8] (1987), Article 450-27, the pad will be provided with a 6" high by 6" wide curb around its perimeter for secondary containment of transformer oil in the event of a leak in the transformer tank. Calculations include provision for containment of worst case 24 hour - 25 year rainfall as well as the transformer oil volume. (See calculation in the Appendix B). Also, in accordance with NEC Article 450 and Article 110, the transformer pad shall be provided with a 8 foot high fence around the pad perimeter and this fence shall be grounded to the facility grounding system. High voltage warning signs shall be clearly posted on all sides the fence. The fence shall have a locked gate to prevent entrance by unauthorized individuals.

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[8]National Electrical Code, NFPA 70, latest revision

Emergency Power - Emergency power generation equipment must be provided for all equipment which must operate under emergency conditions, such as pump motors, trash rakes, controls, and safety related illumination. The emergency generator has therefore been sized to handle the sequential starting loads of the electric pump motors while maintaining adequate voltage levels to operate lighting and control circuits. Once the pump motors have been started, other auxiliary equipment loads may be energized as required. The pumping station operating controls must be designed to provide the necessary sequencing of equipment operation.

Since the station will be unattended, automatic switching mechanisms will be provided for automatic starting of the emergency power generator and switching from commercial to emergency power. Safety interlocks will be provided to prevent operation of the generator with low oil pressure or high temperatures, and to protect the engine and generator from over or under speed operation and overload conditions. Generator installation shall be in accordance with NEC Article 445.

Circuit Protection - Protective devices will be provided in accordance with the latest NEC requirements and any local codes. This includes fusing and circuit breakers for power transformers, lighting transformers, motors, and auxiliary equipment load panels. Specific equipment will be selected as part of the detailed design.

Motor Circuit Protection - Motor circuit protector breakers (magnetic only) and overload protection devices will be provided for all motors. To reduce complexity, across-the-line starters will be used. Motor operating voltages have been selected to allow across-the-line starting without exceeding the maximum allowable surge currents.

Conduit and Wiring System - Conduit installation will utilize corrosion resistant PVC coated, galvanized rigid conduit (GRC) for all exposed wiring runs to protect wiring and for the safety of operating personnel. Fittings will also be protected with a corrosion resistant PVC coating. Conduit systems will be bonded to the station grounding system and provided with supports as required. All conduit systems will contain a separate grounding conductor.

All underground wiring shall be contained in conduit. Underground or concrete encased conduits will be PVC coated GRC. Conduits shall be a buried at a depth in accordance with NEC or local code requirements (whichever is greater), and shall be routed through areas least likely to be subject to equipment anchoring requirements. Underground conduits shall be bonded to the grounding system. All conduit stub-ups shall be threaded GRC and shall be capped to prevent entry of foreign materials until used.

Wiring shall have moisture and heat resistant insulation which is rated for the voltage for which it is used. Wire size must be a minimum of #14 AWG and must be sized in

accordance with the NEC. Procedures for connecting, splicing, terminating conductors will be identified and included in the construction specification.

Grounding - A grounding system in accordance with NEC Article 250 requirements will be provided for all electric equipment, including generator, transformers, busways, motors, panelboards, and lighting and utility systems. A minimum of two grounding rods of sufficient length to reach permanently moist earth will be used, as well as connection to underground water service if available. All equipment frames, enclosures, and conduit systems will be connected to the grounding network in such a way that the resistance to ground does not exceed 3 ohms from any point in the system.

In addition to electrical equipment, fuel storage tank systems shall be grounded in accordance with applicable codes. All electrical equipment within a 10 foot radius of fuel storage tanks shall be explosion proof, rated for Class I, Division 1, Group D.

Intrusion Detection and Vandalism Protection - Intrusion detection requirements include detection of unauthorized entry into the facility and/or structures on the facility. The intrusion detection system alarms will be connected to the facility monitoring and alarm system. The system transmits information to a central facility which is manned around the clock.

The City of Sacramento currently uses a monitoring and alarm system manufactured by Laudis & Gyr, Inc. One system is generally installed per pump sump. The City has upwards of 100 systems presently installed. The present systems have eight channels (N.O. relay contacts), typically assigned as follows:

01 Low Sump
02 SMUD Failure (via phase failure relay)
03 High Sump
04 Low Air Pressure
05 Intrusion Alarms
06 Spare
07 High Water on pump floor
08 AC Failure - (power to alarm panel is off)

The information is transmitted, via leased phone lines or microwave radio link, to the Flood Control & Sewer Division offices on 35th Avenue. The information is decoded, printed out and alarm conditions transmitted to the offices at Sump 2, which are manned around the clock.

Landis & Gyr no longer markets the system now in use by the City of Sacramento for new installations. They do offer a new, expanded capability system which has 16 digital and 4 analog channels. This new system is compatible with the old receiving and decoding system.

NOTE: Any monitoring and alarm system used in new installation should be compatible with the existing system.

The City generally uses switches on doors, gates, and selected movable equipment items. Trip wires are used to detect personnel in the presence of unhoused equipment. Motion detectors are occasionally used, but have been subject to excessive false alarms.

Electrical Equipment Enclosures and Climatization - The present construction concept provides a building to protect the major electrical equipment from exposure to environmental conditions. All equipment located within this building will be provided with NEMA 12 enclosures. All electrical equipment not within this building shall be suitable for outdoor installation, with NEMA 3 or NEMA 4 enclosures.

Climatizing of equipment is not considered to be a major consideration in the Sacramento area. Small electrical heaters will be installed in the electric pump motors and possibly in some of the auxiliary equipment motors to prevent accumulation of moisture. Electrical service and control panels will also contain small heaters to ensure that condensation does not occur. Heaters shall be sized to provide a temperature differential of about 10 degrees above ambient to prevent condensation.

Based on the normal seasonal temperature range, preheating of fuel oil (for the emergency generator and the pumps), does not appear to be a major consideration; however, the manufacturer's recommendations should be followed.

Telephone System - Telephone line installation will be provided by Pacific Bell from an aerial cable running along East Levee Road. The pumping station will be provided with two telephone lines; one for communications and one for intrusion detection systems/monitoring/reporting. In order to incorporate remote monitoring/reporting capability, the phone lines installed will be suitable for touch tone devices.

Area Lighting - Area lighting will be provided for the pumping station facility. Specific design considerations include provision for a minimum illumination of 20 footcandles in areas where there is equipment motion, particularly the areas around the trash removal systems, engine deck, transformer enclosure and at the fuel tank farm. Lower illumination levels (5 to 10 footcandles) are acceptable for

most other areas where the primary purpose is to allow navigation around the equipment. When special lighting is required for maintenance or troubleshooting, portable lighting fixtures can be placed where they provide the best illumination of the equipment being maintained. Emergency lights will be furnished in the electrical and control equipment building.

It should be noted that, since equipment is placed on multiple elevations, shadow effects should be carefully studied and lighting fixture layout designed to minimize shadows caused by structural walls and changes in elevation.

Lightning Protection - Lightning protection in accordance with NFPA 75 is required for the incoming power line (at the source location) and for the area encompassing the pumping station plant. Design and connection of lightning protection devices will be coordinated with the grounding system design. Due to the relatively small area of the pumping stations, (estimated at 150' by 250'), adequate lightning protection may be provided via lightning rods and arrestors on top of area lighting poles. Alternately, lightning protection may be provided using two pole mounted lighting rods (one at each end of the plant), with a conductor stretched between the poles and fitted with lightning arrestors. Since the fuel tank farm is separated from the rest of the plant, it will be provided with separate lightning protection via a lightning rod attached to the tank farm lighting pole and bonded to the plant grounding system.

Electrical System Design Considerations for 100 CFS Pumping Station

Utility Power System Requirements - Power requirements for the 100 cfs pumping station are initially estimated at the following 480 volt loads (Reference Figure 1):

1. Two 20 horsepower, 480 volt, 3 phase pump motors for pump drives, with across the line starting control; total running load 34.4 KW, 43 KVA; starting load (staged to ensure non-simultaneous starting) 134 KVA, 0.8 power factor.
2. Area Lighting, (estimated 10,000 square feet at10-15 fc, high pressure sodium) approximately 2 volt-amp/square foot x 10,000 square feet = 20 KVA.
3. Pump Control System, estimated maximum 20 amps at 120 VAC, single phase = 2.4 KVA.
4. Utility outlets, estimated 20 amps at 120 VAC single phase = 2.4 KVA. (This is adequate to include interior lighting for the control building.
5. Intrusion detection and remote monitoring equipment, estimated at 5 amps at 120 VAC single phase = 0.6 KVA

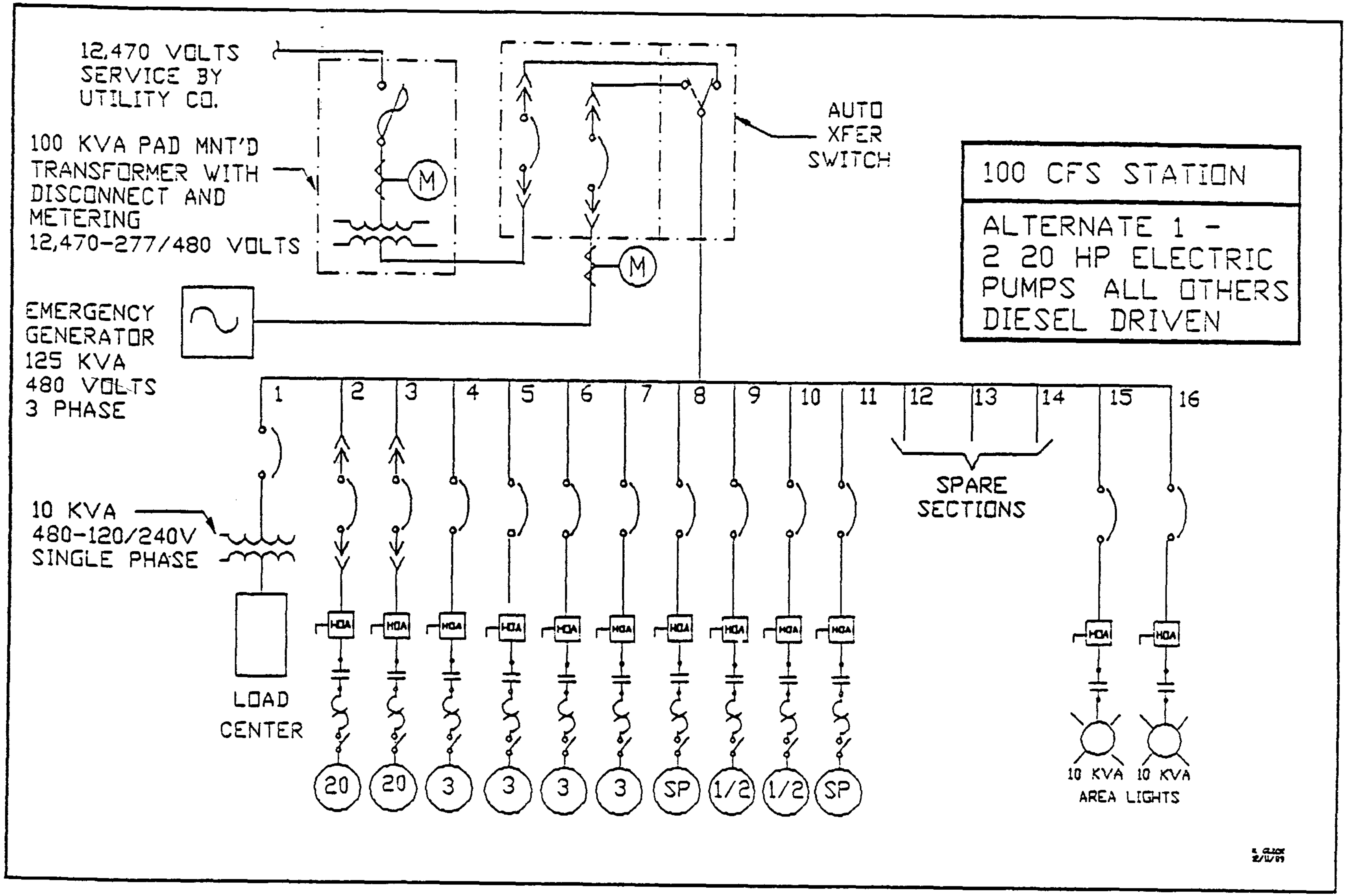

FIGURE 1
100 CFS STATION
SINGLE LINE DIAGRAM

6. Auxiliary equipment:

Trash Rakes, 4 @ 3 hp; total 12 hp,	14.0 KVA
Overhead hoist (if required), 3 hp;	3.5 KVA
Small air compressor, 1 hp;	1.25 KVA
HVAC (if required), 1/2 hp;	0.75 KVA
Miscellaneous Loads	3.5 KVA
Fuel pump, 1/2 hp;	0.75 KVA
Auxiliary Equipment Total load =	23.75 KVA

Total estimated connected load:

43 + 20 + 2.4 + 2.4 + 0.6 + 23.8 = 92 KVA.

Estimated utilization[9]: All loads active except overhead hoist; total load = 92.2 - 3.5 = 88 KVA.

System power feed has therefore been sized for 100 KVA, for a full load utilization of 88%. The local power utility (Sacramento Municipal Utility District) was contacted concerning available power; 12,000 volt service for utilization at 277/480 volt, star connected power was available and could be provided as described above under the electrical utilities section. Above ground service would be provided. The power line cost (including the transformer) for the 100 cfs station is estimated at $111,000.

Note: Since the power line and transformer are normally provided by the utility company at no cost to a government agency, the cost for the installation of the power line is <u>not</u> included in the cost estimates. The above information is for reference in the event that the pumping station becomes privately owned and operated.

Motors under 50 horsepower will not require soft start or reduced voltage starting if using 480 volts. Plant construction costs include provision of a concrete transformer pad and vault for the utility connections. Since the power requirement is low (because most pumps are diesel driven), voltage dip is not a concern except for the sizing of the emergency generator set.

<u>Emergency Power System</u> - The Emergency Power System has been sized to accommodate the starting kva load (staged) of the two electric pump motors with a maximum voltage drop of 15%. Except for starting periods lasting a few seconds, the emergency generator will be adequate to service all loads and separate busses for emergency and normal power are not required. Time delays should be incorporated into the area lighting circuitry to allow all motors to start prior to energizing area lighting loads not required for safety. No other load control requirements are anticipated.

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[9]When operating, estimated at 60 days/year; does not reflect use factor.

Allowing a 15% voltage dip during start periods <u>and insuring that pump motors cannot start simultaneously</u> indicates that a 125 KW, 156 KVA standby rated generator/engine set (approximately 200 hp) should be used. This results in a full load power utilization percentage of 92/156 = 59% of rated standby power during periods when motors are not starting.

1. Since the pumping station is assumed to be unattended under normal conditions, the Emergency Power System will incorporate automatic transfer switchgear. Switch gear will be set up with proper phasing protection and voltage/current monitoring system, interlocked to prevent simultaneous connection of emergency and normal power load relays.
2. Generator engine shall be diesel powered, with self contained cooling system, over/under speed control systems, low oil pressure and high/low temperature alarms, block heater if required, and metering system.
3. Generator engine will be provided with automatic electric (battery powered) start. Battery charging systems will be connected as part of the auxiliary equipment load and energized at all times. Inspection of the battery conditions and charge should be accomplished during regularly scheduled maintenance inspections.
4. Periodic maintenance inspection and monitoring should be routine. Since the station is not attended, either a scheduled maintenance and exercising procedure will be developed or automatic exercising of the generator engine and transfer switches will be provided. If regular testing is scheduled, automatic exercising is not required and could be eliminated.

<u>Control Requirements</u> - In accordance with "General Principles of Pumping Station Design and Layout", COE Publication EM 1110-2-3102, Section 10, pumping station controls will be as simple as possible while providing maximum reliability. Also, both manual and automatic controls will be provided to allow operator override of automatic controls should they fail.

The client has requested that sequence controls be accomplished through simple level detection and sequential pump controls (i.e., relay logic). If the pump control sequence remains fixed, and no alternation is required, than the simplicity of the system logic is such that relay control logic is perfectly satisfactory. Such a logic system is very dependable and easy to maintain, but is of course limited in versatility. If rotation of the pumps is preferred from a maintenance standpoint, or any special sequences are required,

than the control system should probably incorporate a programmable controller to allow the maximum versatility at the lowest cost.

It should be also noted that the control system will be required to provide the operating logic for the diesel driven pumps, even though the electrical system is not required to provide power to the large pump drives.

Level detection may be accomplished via electronic, floats, or pneumatic sensing methods. Engineering manual EM 1110-2-3101, 1962 (pg 46) states that electronic probes have not been proven reliable for this application, and recommends the use of mechanical floats, with appropriate material selection to minimize corrosion. Since the publication of EM 1110-2-3101 (1962), significant improvements to electronic sensor technology (including available probe materials) have occurred which have increased electronic probe reliabilities, and they should be considered. Pneumatic sensing of head elevation is preferred by the City of Sacramento and eliminates the problems associated with moving parts (except for the pressure switches), but requires compressed air. The electronic probes and signal conditioning devices vs. pneumatic sensing including air compressor(s) should be compared for initial cost, maintenance consideration, and reliability (the major consideration) before a final decision is made.

Repeatability and accuracy of the level detection devices are not critical parameters, except that the system as designed will require the capability to sense 11 - 12 distinct levels within a range of 24 inches. For pressure switches working in inches of water, this requires sensitivity to within ± 0.5 inches of water pressure, which would be quite sensitive to pressure fluctuations of the supply compressor(s) or other effects such as turbulence. Similar considerations apply to the electronic devices. For either method, accuracy and repeatability requirements may be easily achieved.

Electrical System Design Considerations for 400 CFS Pumping Station

The electrical design of the 400 cfs station is nearly identical to the 100 cfs station, except that the two small pumps will be increased to 100 horsepower each. All other pumps are diesel engine driven. The increased horsepower of the two small pumps affects only the electrical power system requirements, including the power line and transformer and the emergency generator.

<u>Utility Power System Requirements</u> - Power requirements for the 400 cfs pumping station are initially estimated at the following 480 volt loads (Reference Figure 2):

1. Two 100 horsepower, 3 phase pump motors for pump drives, total running load 168 KW, 210 KVA; motor starting load (staged to ensure non-simultaneous starting) 670 kva (each motor), 0.8 power factor.

   Note: All other loads indicated below are identical to the 100 cfs station.

2. Area Lighting, approximately 2 volt-amp/square foot (high pressure sodium, 15-20 fc) x 10000 estimated square feet = 20 KVA.
3. Pump Control System, estimated maximum 20 amps at 120 VAC, single phase = 2.4 KVA.
4. Utility outlets 20 amps at 120 VAC, 1 ϕ, = 2.4 KVA. (This would probably be adequate to include interior lighting for a 500 square foot building if installed).
5. Intrusion detection and remote monitoring equipment, estimated at 5 amps at 120 VAC, 1 ϕ, = 0.6 KVA
6. Auxiliary equipment:

Trash Rakes, 4 @ 3 hp; total 12 hp,	14.0 KVA
Overhead hoist (if required), 3 hp;	3.5 KVA
Small air compressor, 1 hp;	1.25 KVA
HVAC (if required), 1/2 hp;	0.75 KVA
Miscellaneous Loads	3.5 KVA
Fuel pump, 1/2 hp;	0.75 KVA
Auxiliary Equipment Total load =	23.75 KVA

Total estimated connected load:
210 + 20 + 2.4 + 2.4 + 0.6 + 23.8 = 259 KVA.

Estimated utilization[10]: All loads active except overhead hoist; total load = 259 - 3 = 256 KVA.

System power feed has therefore been sized for 250 KVA, for a utilization of 259/250 = 104%. The local power utility (Sacramento Municipal Utility District) was contacted concerning available power; 12,000 volt service was available and could be provided by overhead service. The power line cost (including the transformer) for the 400 cfs station is estimated at $118,000.

[10]When operating, estimated at 60 days/year; does not reflect use factor.

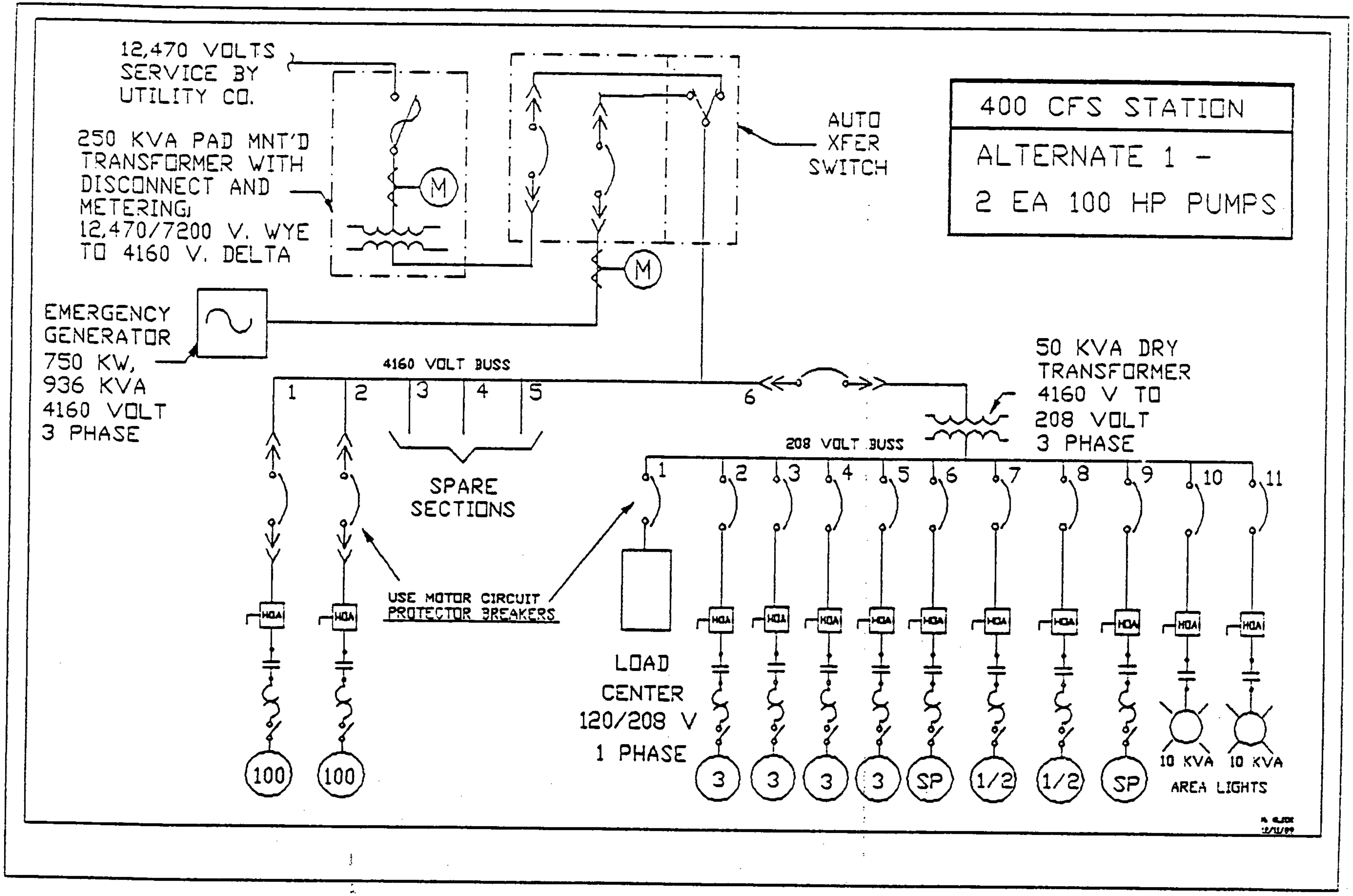


FIGURE 2
400 CFS STATION
SINGLE LINE DIAGRAM

Note: Since the power line and transformer are normally provided by the utility company at no cost to a government agency, the cost for the installation of the power line is <u>not</u> included in the cost estimates. The above information is for reference in the event that the pumping station becomes privately owned and operated.

To reduce surge current requirements during across-the-line starting of the 100 horsepower motors, the 12,400 volt, 250 KVA incoming power will be stepped down to 4160 volts for operation of the 100 horsepower motors. A second 50 KVA transformer will further reduce the 4160 volts to 208 volts for operation of the remainder of the facility. All other motors will operate at 208 volts three phase, and a 120/208 volt single phase load center will be provided for general equipment service.

<u>Emergency Power System</u> - Emergency power must be provided to allow operation of the 100 horsepower pumps, the trash rakes, pump drive (including both diesel and electric) control systems, and the area lighting during utility failure.

The Emergency Power System for the 400 cfs pumping station has therefore been sized to accommodate the starting kva load (staged) of the two electric pump motors with a maximum voltage drop of 15% to maintain adequate voltage levels for operation of necessary lighting and control circuits. Once the pump motors have been started, other auxiliary equipment loads may be energized as required. The pumping station operating controls must be designed to provide the necessary sequencing of equipment operation.

Except for starting periods lasting a few seconds, the emergency generator will be adequate to service all loads and separate busses for emergency and normal power are not required. Time delays should be incorporated into the area lighting circuitry to allow all motors to start prior to energizing area lighting loads not required for safety. No other load control requirements are anticipated.

Allowing a 15% voltage dip during start periods indicates that a 750 KW, 938 KVA standby rated generator/engine set should be used. This results in a full load power utilization percentage of 259/938 = 28% of rated standby power during periods when the pump motors are not starting.

1. Since the pumping station is assumed to be unattended under normal conditions, the Emergency Power System will incorporate automatic transfer switchgear. Switch gear will be set up with proper phasing protection and voltage/current monitoring system, interlocked to prevent simultaneous connection of emergency and normal power load relays.

2. Generator engine shall be diesel powered, with self contained cooling system, over/under speed control systems, low oil pressure and high/low temperature alarms, block heater if required, and metering system.
3. Generator engine will be provided with automatic electric (battery powered) start. Battery charging systems will be connected as part of the auxiliary equipment load and energized at all times. Inspection of the battery conditions and charge should be accomplished during regular maintenance inspections.
4. Periodic maintenance inspection and monitoring should be routine. Since the station is not attended, either a scheduled maintenance and exercising procedure will be developed or automatic exercising of the generator engine and transfer switches will be provided. If regular testing is scheduled, automatic exercising is not required and could be eliminated.

Control Requirements - Control requirements for the 400 cfs pumping station are identical to the 100 cfs station.

Electrical System Design Considerations for 700 CFS Pumping Station

The electrical design of the 700 cfs station is nearly identical to the 400 cfs station, except that the two small pumps will be increased to 200 horsepower each. All other pumps are diesel engine driven. The increased horsepower of the two small pumps affects only the electrical power system requirements, including the power line and transformer and the emergency generator.

Utility Power System Requirements - Power requirements for the 700 cfs pumping station are initially estimated at the following loads (reference Figure 3):

1. Two 200 horsepower, 3 phase pump motors for pump drives, total running load 328 KW, 410 KVA; starting load (staged to ensure non-simultaneous starting) 1340 kva (each motor), 0.8 power factor.

   Note: All other loads indicated below are identical to both the 100 cfs and 400 cfs station.

2. Area Lighting, approximately 2 volt-amp/square foot (high pressure sodium, 15-20 fc) x 10000 estimated square feet = 20 KVA.
3. Pump Control System, estimated maximum 20 amps at 120 VAC, single phase = 2.4 KVA.

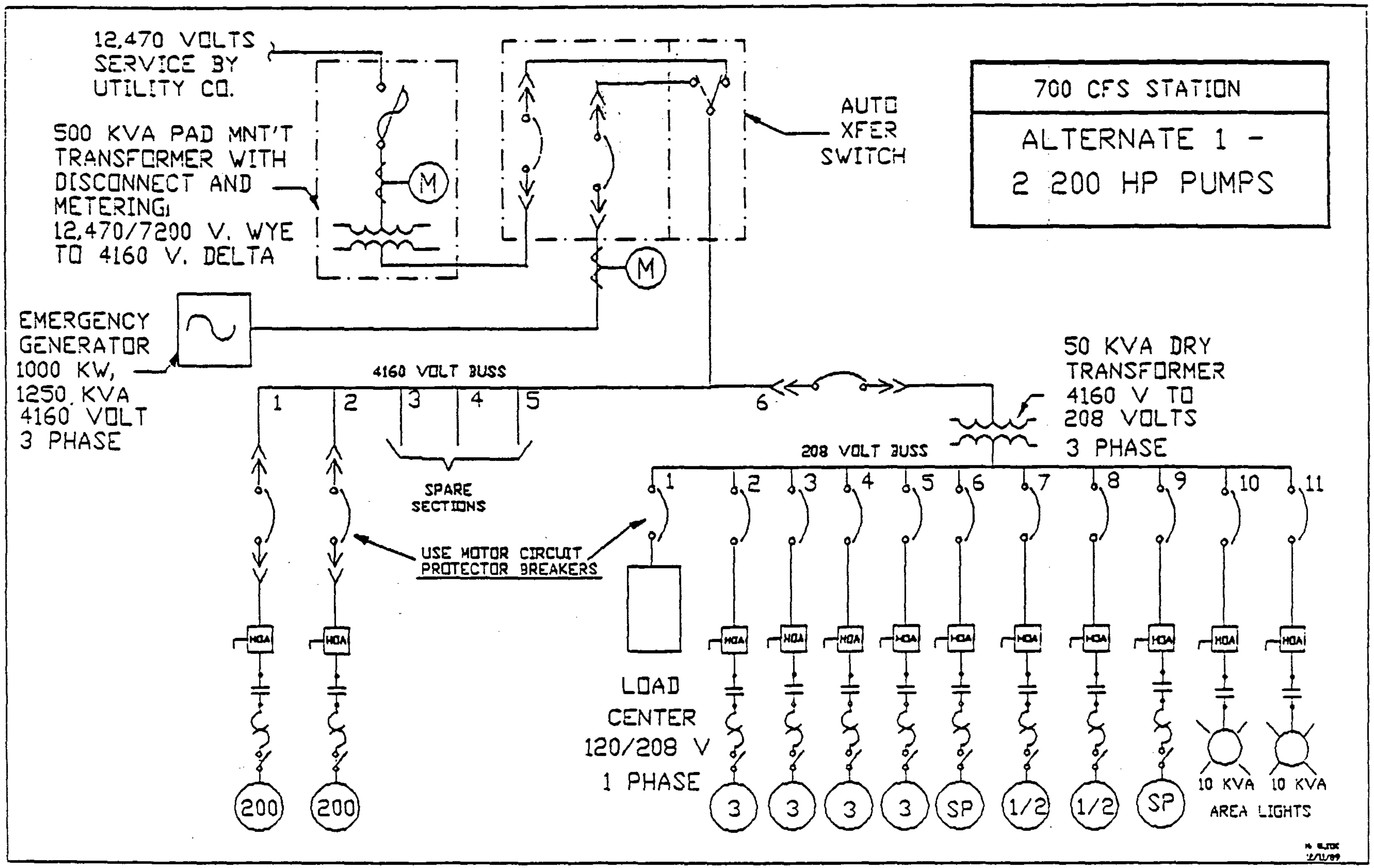


FIGURE 3
700 CFS STATION
SINGLE LINE DIAGRAM

4. Utility outlets 20 amps at 120 VAC, 1 phase = 2.4 KVA. (This would probably be adequate to include interior lighting for a 500 square foot building if installed).
5. Intrusion detection and remote monitoring equipment, estimated at 5 amps at 120 VAC, 1 phase = 0.6 KVA
6. Auxiliary equipment:

Trash Rakes, 4 @ 3 hp; total 12 hp,	14.0 KVA
Overhead hoist (if required), 3 hp;	3.5 KVA
Small air compressor, 1 hp;	1.25 KVA
HVAC (if required), 1/2 hp;	0.75 KVA
Miscellaneous Loads	3.5 KVA
Fuel pump, 1/2 hp;	0.75 KVA
Auxiliary Equipment Total load =	23.75 KVA

Total estimated connected load:

410 + 20 + 2.4 + 2.4 + 0.6 + 23.8 = 459 KVA.

Estimated utilization[11]: All loads active except overhead hoist; total load = 459 - 3 = 456 KVA.

System power feed has therefore been sized for 500 KVA, for a full load utilization of 459/500 = 92%. The local power utility (Sacramento Municipal Utility District) was contacted concerning available power; 12,000 volt service was available and could be provided by overhead service. The power line cost (including the transformer) for the 400 cfs station is estimated at $135,000.

Note: Since the power line and transformer are normally provided by the utility company at no cost to a government agency, the cost for the installation of the power line is not included in the cost estimates. The above information is for reference in the event that the pumping station becomes privately owned and operated.

To reduce surge current requirements during across-the-line starting of the 100 horsepower motors, the 12,400 volt, 500 KVA incoming power will be stepped down to 4160 volts for operation of the 200 horsepower motors. A second 50 KVA transformer will further reduce the 4160 volts to 208 volts for operation of the remainder of the facility. All other motors will operate at 208 volts three phase, and a 120/208 volt single phase load center will be provided for general equipment service.

Emergency Power System - Emergency power must be provided to allow operation of the 200 horsepower pumps, the trash rakes, pump drive (including both diesel and electric) control systems, and the area lighting during utility failure.

[11]When operating, estimated at 60 days/year; does not reflect use factor.

The Emergency Power System for the 700 cfs pumping station has therefore been sized to accommodate the starting KVA load (staged) of the two electric pump motors with a maximum voltage drop of 15% to maintain adequate voltage levels for operation of necessary lighting and control circuits.

Once the pump motors have been started, other auxiliary equipment loads may be energized as required. The pump station operating controls must be designed to provide the necessary sequencing of equipment operation.

Except for starting periods lasting a few seconds, the emergency generator will be adequate to service all loads and separate busses for emergency and normal power are not required. Time delays should be incorporated into the area lighting circuitry to allow all motors to start prior to energizing area lighting loads not required for safety. No other load control requirements are anticipated.

Typical starting load for a 200 hp motor is about 1356 KVA. Allowing a 15% voltage dip during start periods indicates that a 1000 KW, 1250 KVA generator/engine set should be used. This results in a full load standby power utilization percentage of 459/1250 = 37% during periods when the pump motors are not starting.

1. Since the pumping station is assumed to be unattended under normal conditions, the Emergency Power System will incorporate automatic transfer switchgear. Switch gear will be set up with proper phasing protection and voltage/current monitoring system, interlocked to prevent simultaneous connection of emergency and normal power load relays.
2. Generator engine shall be diesel powered, with self contained cooling system, over/under speed control systems, low oil pressure and high/low temperature alarms, block heater if required, and metering system.
3. Generator engine will be provided with automatic electric (battery powered) start. Battery charging systems will be connected as part of the auxiliary equipment load and energized at all times. Inspection of the battery conditions and charge should be accomplished during regular maintenance inspections.
4. Periodic maintenance inspection and monitoring should be routine. Since the station is not attended, either a scheduled maintenance and exercising procedure will be developed or automatic exercising of the generator engine and transfer switches will be provided. If regular testing is scheduled, automatic exercising is not required and could be eliminated.

Control Requirements - Control requirements for the 700 cfs pumping station are identical to both the 100 cfs and 400 cfs station.

Alternative Pump Drive Configurations

A total of three configurations were evaluated during the pumping station design. These include mixed electric and diesel driven pumps as described above, a configuration consisting of all diesel pump drives, and a configuration consisting of all electric pump drives.

Each of these configurations results in different power system requirements, primarily in the power line and transformer sizing, the sizing for the emergency generator, and the selection of pump motor operating voltages.
Table N-2-2 on the following pages summarizes the sizing requirements for the service capacity, transformer sizing and the emergency generator sizing required for each of the three designs.

ARCHITECTURAL DESIGN

The architectural design of the pumping station will be consistent with the guidance provided in EM 1110-2-3103, Architectural Design of Pumping Station. A site visit with the City of Sacramento Sewers and Flood Control Division to various existing pumping stations that it operates revealed that their preferred construction materials are consistent with those of EM 1110-2-3103.

The facility will be an above ground pumping station located in an isolated industrial/rural area. The station will be the outdoor type with provisions for providing a superstructure (building) at a later date should residential pressures justify such treatment to reduce noise. Generally the station will be similar to those shown on Plates Nos. 2, 3, 7, and 8 of EM 1110-2-3103, excepting the superstructure. The proposed station layout including the bypass channel and diversion dike will be most similar to the Mill Creek pumping station shown on Plate No. 8, again excepting the superstructure.

TABLE N-2-2
COPMARISON OF ELECTICAL EQUIPMENT
SIZING FOR ALTERNATE PUMP DRIVE CONFIGURATIONS

	Mixed Drives	All Diesel Drives	All Electric Drives
100 CFS Station:			
Connected Load	92 KVA	54 KVA	277 KVA
Service Capacity	100 KVA	75 KVA	300 KVA
Generator Capacity	125 KW 156 KVA	80 KW 100 KVA	275 KW 344 KVA
Motor Operating Voltages			
Pumps	480 VAC		4160 VAC
Other	480 VAC	480 VAC	208 VAC
400 CFS Station:			
Connected Load	259 KVA	54 KVA	1021 KVA
Service Capacity	250 KVA	75 KV	1000 KVA
Generator Capacity	750 KW 938 KVA	80 KW 100 KVA	1000 KW 1250 KVA
Motor Operating Voltages			
Pumps	4160 VAC		4160 VAC
Other	208 VAC	480 VAC	208 VAC

Table N-2-2 (Continued)

	Mixed Drives	All Diesel Drives	All Electric Drives
700 CFS Station:			
Connected Load	459 KVA	54 KVA	1628 KVA
Service Capacity	500 KVA	75 KV	1500 KVA
Generator Capacity	1000 KW 1250 KVA	80 KW 100 KVA	1500 KW 1875 KVA
Motor Operating Voltages			
Pumps	4160 VAC		4160 VAC
Other	208 VAC	480 VAC	208 VAC

A pre-engineered steel building, 24 ft. by 26 ft., will be provided to house the motor control center and standby diesel electric generator. The building will also provide space for a small office including a desk, chair, locker, workbench and other similar items as needed.

The type of construction for the substructure will be cast-in-place reinforced concrete. The concrete deck will support the mechanical and electrical components and also provide a roadway. The engines will be exposed.

Other miscellaneous construction details are: Floor beams will be reinforced concrete. Painting will be in compliance with Corps guide specifications. Interior office and exterior operations lighting will be provided. No heating equipment included, nor toilet facilities. Provisions for cleaning out the sumps after floods shall be provided by access hatches. Other than graveled surfaced adjacent to the station and seeding of new slopes, no landscaping is planned. Perimeter fencing and gates will be provided. Removable steel handrails and guardrails will be provided where persons might fall such as the forebay and afterbay deck edges. Grating over the discharge flap gates will be provided for easy observation and access. A small crane (2 ton) will be supplied with the engines, motors, and other equipment for routine maintenance. The crane will be manually pushed on large wheels with locking hubs. Lifting will be through a motorized hoist. A public sign identifying the structure is anticipated.

STRUCTURAL DESIGNS AND STABILITY

General Civil Design

Loading - "Normal loading for civil structures, unless otherwise defined in particular criteria, shall be:

1. Dead Loads - The dead loads will consist of the actual weight of the structure, permanent construction and fixtures.
2. Hydrostatic Loads due to maximum headwater and tailwater levels and water levels during the pump start-up and shut-down.
3. Hydrostatic Uplift assumed effective over 100 percent of the base area of the structure with a straight line variation between points of known pressure.
4. Live Loads - Live loads will consist of the weights of the machinery, equipment, stored materials, personnel, mobile crane, impact from any of the foregoing loads, normal pump thrust and rotational loads, and wind loads.

Feature	Loading (psf)
Station Floor & Access Hatches	250
Forebay Deck Access Road (AASHTO H20)	640 # per lane & 18,000 # Concentrated load.
Afterbay Deck & Grates	250
Pre-Engineered Steel Building	250
Wind (Uniform building Code)	33 (Horiz.) 18 (Vert.)

"Unusual" or "extreme" loading for civil structures, unless otherwise defined in particular criteria, shall be:

5. Maximum Earthquake Loads and hydrodynamic earthquake forces.

Site acceleration - 0.22 g

Design - The working stress design method designated as Alternate Design Method in ACI Code 318-83 Appendix B shall be used for design of the pumping substructure. Concrete compressive strength shall be 3000 psi at 28 days. Reinforcing steel shall be grade 60. The entire substructure shall be monolithic with no contraction joints.

Plant Stability

Loading Conditions - The pumping station must be stable and all its components must be capable of resisting induced stresses from all expected operating conditions. The pumping station shall be analyzed for stability and structural integrity for the following operating conditions.:

1. <u>Normal Operation</u> - No pumps operating with afterbay and forebay water surface elevation from 23.4 to 28.
2. <u>Maintenance Condition</u> - One pump sump empty, water surface elevations same as in Case 1 above.
3. <u>Construction</u> - Backfill without hydrostatic load, and no equipment.
4. <u>Earthquake</u> - Case 1 plus seismic forces.
5. <u>Flood Condition</u> - All pumps operating with afterbay water surface elevation of 39 and forebay water surface elevation from 28 to 39.

<u>Design Assumptions</u> -

1. <u>Uplift</u> - The entire pumping station base is subject to uplift pressure. The uplift pressure is assumed to be linearly variable between the forebay and afterbay water pressures.
2. <u>Foundation Material</u> - The foundation is composed of sandy clays. The coefficient of friction for the shear-friction computation is 0.45,. per the geotechnical section.
3. <u>Backfill</u> - The backfill shall be composed of silty sand and sandy clay obtained from the pumping station site excavation. This material is assumed to have the following characteristics:

Dry Density	110 lb/ft^3
Void Ratio	0.50
Specific Gravity of Solids	2.65
Angle of Repose	30°
Pressure Coefficient (at rest)	0.5
Pressure Coefficient (active)	0.32
Pressure Coefficient (passive)	3.14

<u>Stability Requirements</u> - The pumping station as a whole must satisfy the requirements tabulated below:

Load Condition	Flotation Safety Factor	Sliding Safety Factor	Overturning Safety Factor
1. Normal Operation	1.2	2.0	1.2
2. Maintenance	1.1	2.0	1.2
3. Construction	-	1.5	1.1
4. Earthquake	1.1	1.5	1.1
5. Flood Control	1.1	1.5	1.1

SECTION III - SOILS AND FOUNDATION DESIGN

GENERAL

The site of the proposed Pumping Station is shown on Sheet 1, Appendix D. This section of the report has been prepared to summarize the data obtained during this geotechnical portion of the study and to present conclusions and recommendations based on the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to design are included.

FIELD INVESTIGATION

The field exploration was conducted on November 13, 1989. Two borings were drilled to depths of 25 feet at the locations shown on Sheet 1 to explore subsurface conditions at the site. Location of the borings were estimated by pacing from features shown on the site plan. Top of boring hole, elevations were estimated at 29.0 from contours as shown on the site plan.

Borings were advanced using 7 inch diameter, continuous flight hollowstem augers. Samples were obtained by using a 2 inch inside diameter California Sampler. The sampler was advanced by blows from a 140 pound hammer falling 30 inches. The number of blows required to drive the sampler 12 inches, is known as the penetration resistance or "N" value. The depth at which samples were taken and the penetration resistance values are shown on the Logs of Borings, Sheet 4. The borings were logged by a representative of Chen-Northern, Inc.

LABORATORY TESTING

Samples obtained during the field investigation were taken to the laboratory where they were observed and visually classified in accordance with ASTM D-2487 which is based on the Unified Soil Classification System. Representative samples were selected for testing to determine the engineering and physical properties of the soils in general accordancewith ASTM or other approved procedures. The following list outlines the tests conducted and the soil characteristics that were to be determined.

Tests Conducted:	To Determine:
Grain-size Distribution Sheet 4, Figures 4 and 5 & Table N-2-3	Size and distribution of soil particles; that is, clay, silt, sand and gravel.

Atterberg Limits Sheet 4 & Table N-2-3	A method of describing the affect of varying water content on the consistency of fine-grained soils.
Natural Moisture Content Sheet 4 & Table N-2-3	Moisture content representative of field conditions at the time samples were taken.
Natural Dry Density Sheet 4 & Table N-2-3	Dry unit weight of samples representative of in place conditions.
Consolidation/Swell Figure 6	The amount that a soil sample compresses with loading and the influence of wetting on its behavior. For use in settlement analysis, determining expansive potential and foundation design.
Unconfined Compression Sheet 4 & Table N-2-3	General soil strength properties.

Results of all laboratory tests are summarized on the following Tables N-2-3 and Figures 4, 5, and 6 as indicated above.

These data along with the field information was used to prepare Logs of Borings as shown on Sheet 4, Appendix D.

SUBSURFACE CONDITIONS

The subsurface profile encountered within the borings consists of a silty sand extending to depths of between 11 and 12 feet below existing grade. Underlying the silty sand, interlayered clayey sand and sandy clay were encountered to a depth of between 21 feet in Boring B-1 and to the maximum boring depth of 25 feet in Boring B-2. In Boring B-1, silty sand was encountered beneath the clayey soils to the maximum depth.

The sand is silty and is considered to be very dense with penetration resistance values ranging from 70 blows per foot to refusal. Refusal is considered to be greater than 50 blows with less than 6 inches of penetration. The moisture content of the silty sand ranges from 8 to 22% with in place densities ranging from 81 to 116 pounds per cubic foot. The soil is slightly moist to moist and brown in color.

The clayey sand and sandy clay is very dense to hard with penetration resistance values ranging from 49 to approximately 82 blows per foot. The natural moisture content of the clayey sand and sandy clay varies from 16 to 23%. The in-situ natural dry density, ranges from 96 to 113 pounds per cubic feet.

California Pumping Station
Job No. 5-115-90

Chen Northern, Inc.

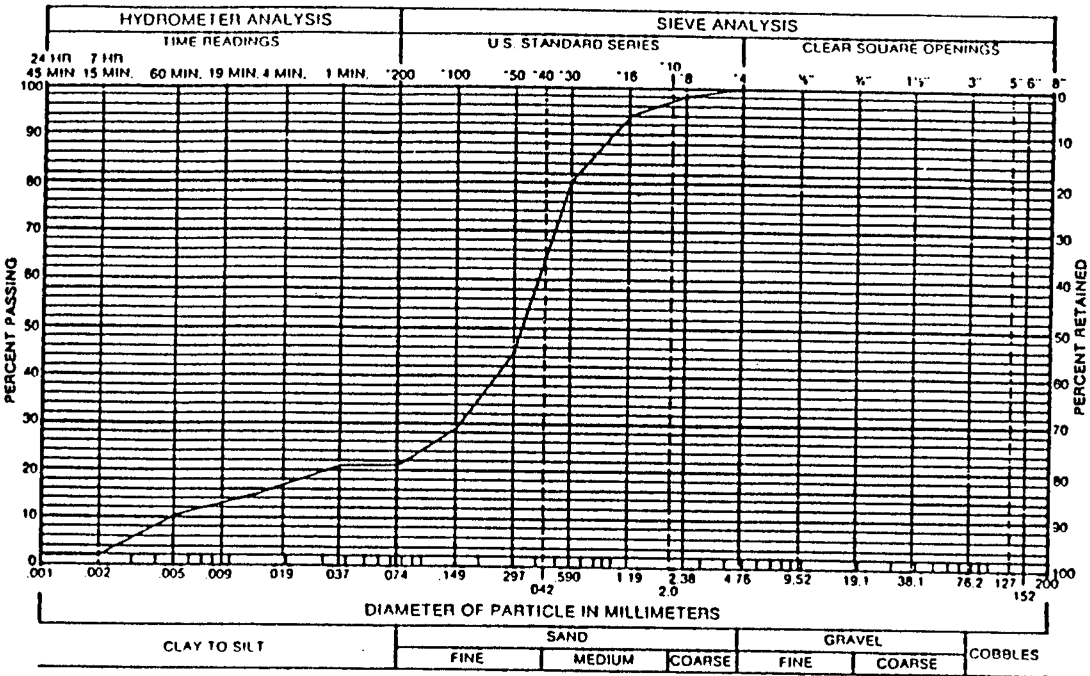
TABLE 3

SUMMARY OF LABORATORY TEST RESULTS

SAMPLE LOCATION		NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (PCF)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (PSF)		SOIL OR BEDROCK TYPE
HOLE	DEPTH (FEET)			GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)			
B-1	2	8	116	0	79	21					Silty Sand (SM)
	4	15	81			20					Silty Sand (SM)
	9	10	116	0	78	22					Silty Sand (SM)
	14	16	113			41	34	17			Clayey Sand (SC)
	19	23	96			73	37	15	4200		Sandy Clay (CL)
	24	15	116			39					Silty Sand (SM)
B-2	2	15	98	0	71	29					Silty Sand (SM)
	4	14	74			51			3540		Sand and Clay (SC-CL)
	9	22	92	0	72	28			2180		Silty Sand (SM)
	14	18	107			64	38	20			Sandy Clay (CL)
	19	16	109			41	31	13			Clayey Sand (SC)
	24	16	112			39					Clayey Sand (SC)

TABLE 3
SUMMARY OF LABORATORY
TEST RESULTS

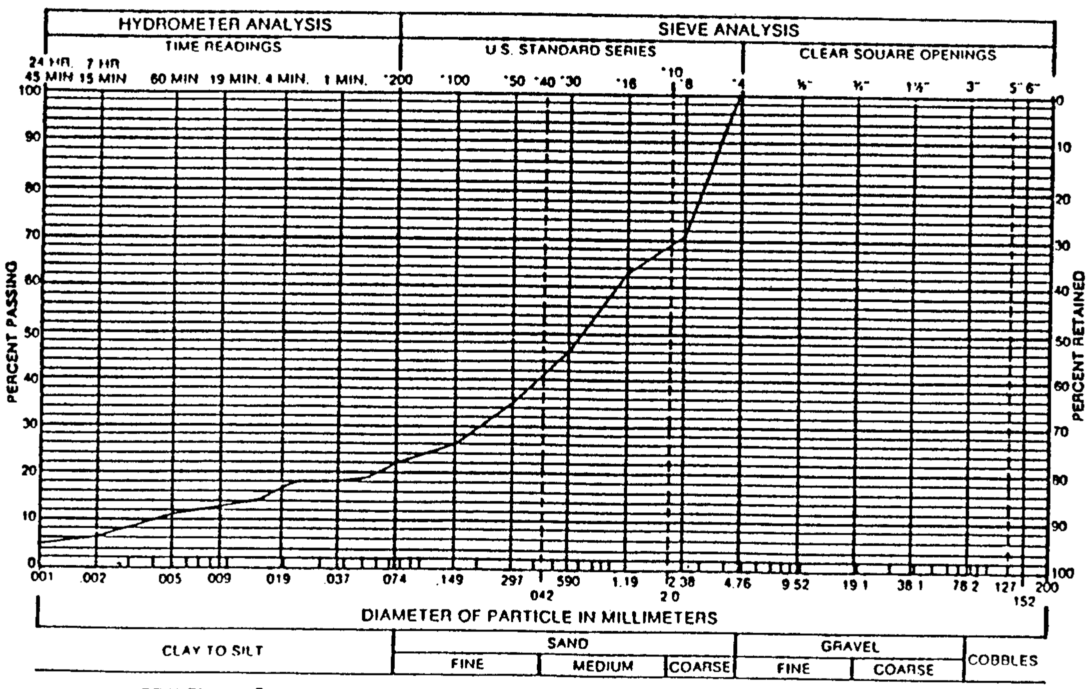
Chen-Northern, Inc.



GRAVEL 0 % SAND 79 % SILT AND CLAY 21 %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Silty Sand (SM) FROM B-1 at 2 feet



GRAVEL 0 % SAND 78 % SILT AND CLAY 22 %

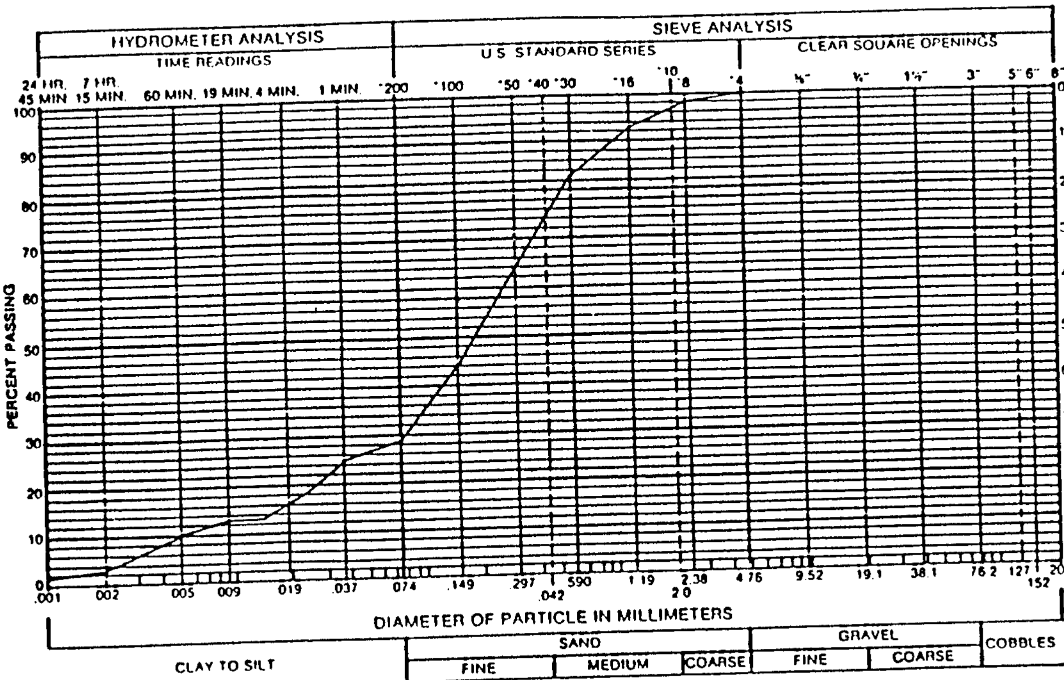
LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Silty Sand (SM) FROM B-1 at 9 feet

GRADATION TEST RESULTS

FIGURE 4
GRADATION TEST RESULTS FOR BORINGS B-1

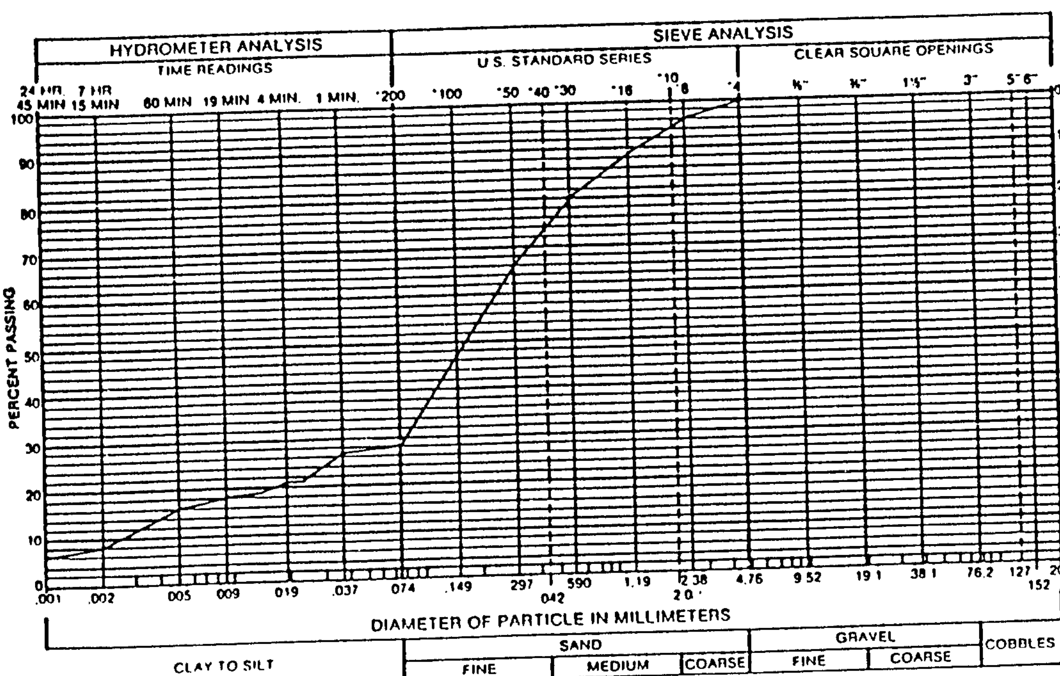
Chen Northern, Inc.



GRAVEL 0 % SAND 71 % SILT AND CLAY 29 %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Silty Sand (SM) FROM B-2 at 2 feet



GRAVEL 0 % SAND 72 % SILT AND CLAY 28 %

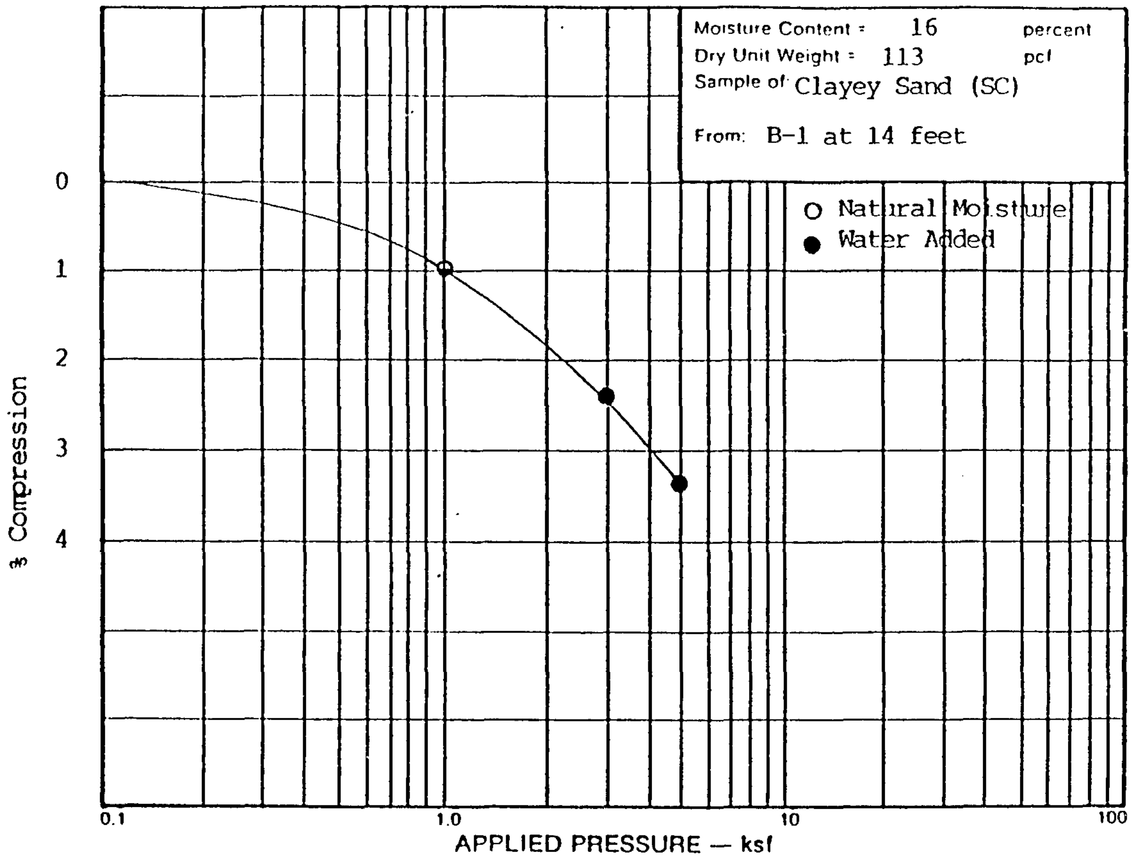
LIQUID LIMIT % PLASTICITY INDEX %

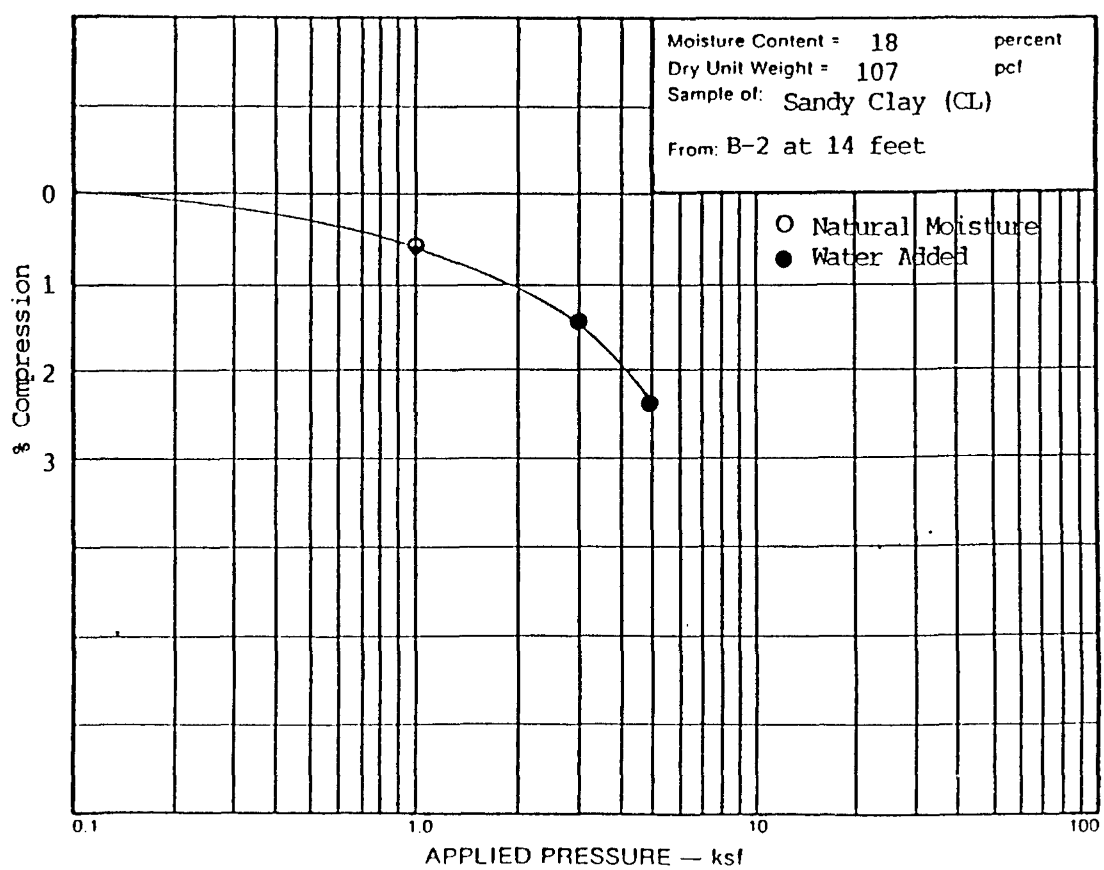
SAMPLE OF Silty Sand (SM) FROM B-2 at 9 feet

GRADATION TEST RESULTS

FIGURE 5
GRADATION TEST RESULTS FOR BORINGS B-2

Chen-Northern, Inc.





SWELL-CONSOLIDATION TEST RESULTS

FIGURE 6
SWELL-CONSOLIDATION TEST RESULTS

No ground water was observed within the borings at the time of drilling. Due to the fine grain nature of the soils, it is possible that water levels could have been higher than what was observed, but may not have yet reached a steady state condition to allow for measurement. Numerous factors contribute to the fluctuation of ground water levels and the evaluation of such factors is beyond the scope of this report.

SITE GRADING

General

The use of on site soils for fill, either as structural fill beneath the foundations or for the construction of dikes, will be appropriate. The excavation of soil to the depths anticipated can probably be accomplished with most heavy duty, earth excavating equipment.

Recommendations

(1) All top soil, organic material, existing fill or other debris should be removed from the location of the proposed facilities.

(2) All fill and backfill should be approved by the geotechnical engineer, placed in uniform lifts and compacted to at least 95% of the maximum dry density as measured by ASTM D1557 within 2% of optimum moisture.

(3) Any locations within the proposed construction area which are soft or which are disturbed during construction activities, should be removed and replaced with compacted structural fill.

(4) Due to the nature of the clayey and silty soils encountered at the site, special care should be taken to avoid soil disturbance during construction.

(5) Permanent cut slopes in the existing soils should be no steeper than 2 horizontal to 1 vertical and no steeper than 2.5 to 1 where seepage pressures will exist. For erosion protection, slopes may need to be flatter. The stability of these slopes should be re-examined in greater detail during final design.

(6) Temporary cut slopes should be no steeper than 1 to 1 where the slope is less than 10 feet in height. Cuts with greater height or where groundwater is encountered should be no steeper than 1.5 to 1.

FOUNDATIONS

General

The lateral dimensions of the proposed alternative facilities will be approximately 75 to 85 feet in length with a width varying from 73 to 109 feet. The flow line of the

existing channel is at an elevation of approximately 23.4 and it is anticipated that the bearing surface of the foundation for the proposed facility will be at an elevation of approximately 15.0. The facility will be supported by a continuous mat underlying the entire facility. It is anticipated that the maximum loading from the facility will be approximately 2000 lbs. per square foot over the entire area. Based on the one dimensional consolidation theory, the water level and soil characteristics encountered and using a long term bearing pressure of 1500 pounds per square foot for the entire facility, we estimate the total settlements will be approximately on the order of 1 3/4 inches near the center and approximately 2/3 of an inch around the outside edge of the structure. It is estimated that 75% of the total settlement will be completed by the end of construction.

Appurtenant structures constructed at the site should be founded on individual spread footings designed for a maximum allowable bearing pressure of 3,000 pounds per square foot. The settlement of such structures is anticipated to be less than approximately one inch.

Recommendations

(1) Mats and footings should be supported on undisturbed natural soil or on compacted structural fill.
(2) Footings should have a minimum depth of embedment of 12 inches.
(3) Footings should have a minimum width of 18 inches for continuous footings and 24 inches for isolated spread footings.
(4) The continuous foundation walls should be reinforced top and bottom to span an unsupported length of at least 10 feet.

FOUNDATIONS AND RETAINING WALLS

General

Foundation wall and other retaining structures are subjected to horizontal loading due to lateral earth pressure. This pressure is a function of natural and backfill soil types and acceptable wall movement which effect soil strain and mobilize the shear strength of the soil. More soil movement is required to develop greater internal shear strength and lower the lateral pressure on the wall. Soil strain and allowable wall rotation must be greater to mobilize full strength and reduce lateral pressures for clay soils than for cohesionless granular soils. The distribution of lateral earth pressures on the structure depends on the soil type and wall movements. In most cases a triangular, pressure distribution is satisfactory for design and is usually represented as an equivalent fluid weight.

Recommendations

(1) Foundation walls and other retaining walls which are laterally supported and can be expected to only undergo a slight amount of deflection should be designed for a lateral pressure computed on the basis of an equivalent fluid weight of 50 pounds per cubic foot (equivalent to an at rest pressure coeffieicnt of 0.5). Where these soils may become saturated, an equivalent fluid pressure of 90 pounds per cubic foot should be used in design.
(2) Retaining structures which can deflect sufficiently to mobilize full active pressure conditions should be designed for a lateral earth pressure computed on the basis of an equivalent fluid weight of 35 pounds per cubic foot (equivalent to an active pressure coefficient of 0.32). When backfill soils become saturated, design should be based on an equivalent fluid weight of 80 pounds per cubic foot.
(3) The lateral resistance of footings placed on natural soil or on properly compacted structural fill should be designed using a coefficient of friction of 0.45. Passive earth pressures can be calculated using an equivalent fluid weight of 350 pounds per cubic foot (equivalent to a passive pressure coefficient of 3.14). Where backfill soils become saturated, passive pressure should be calculated based on equivalent fluid weight of 190 pounds per cubic foot.
(4) The values recommended above assume a mobilization of ultimate soil strength. Conventional safety factors used in structural analysis for items such a overturning and sliding should be used in the design to limit strain which will occur at ultimate strength particularly in the case of passive resistance.
(5) Foundation and retaining wall backfill placed against the side of footings to resist lateral loads should be placed and compacted according to the criteria presented in Section 3-5 Site Grading of this report. Care should be taken not to over compact the backfill since this could cause excessive lateral pressure on the wall.

DIKES

General

The use of on-site soils is considered appropriate for the construction of dikes. The on-site soils, however, are considered to be highly erodible.

Recommendations

(1) Levees and dikes should be constructed to a slope of no greater than 2.5 horizontal to 1 vertical, where seepage

pressures within the dike could exist from possible sudden drawdowns. Side slopes of 2 horizontal to 1 vertical can be used where seepage pressures will not exist. Flatter slopes may be required for erosion protection. The stability of these slopes should be re-examined in greater detail during final design.

(2) The side slopes of dikes and levees which will be subject to erosion from the movement of water should be protected from erosion by mulching, fertilizing and seeding with mixed grasses.

(3) It is anticipated where there is a potential for high water velocities near the inlet and outlet portions of the pump station that increased protection from erosion of the silty sands will be required through the use of rip-rap. In those areas where rip-rap is to be placed, a graded filter should be placed between existing soils and the rip-rap for erosion protection. This filter layer should have a minimum thickness of 8 inches and should meet the following gradation requirements:

Sieve Size	Percent Passing
3/4 inch	100%
#4	50-75%
#16	10-40%

It is anticipated that the rip-rap will be evenly graded with a maximum size of approximately 6 inches and a minimum size of 3/4 of an inch.

SEEPAGE

General

It is anticipated that the permeability of the natural soils at the site is low to very low due to the high percentage of fines contained in the soil. The soil permeability, as referenced in "Foundation Analysis & Design" by Joseph E. Bowles, P. 38, McGraw-Hill, 1982, is estimated to be in the following ranges:

Soil	Permeability
Silty Sand	10 to 100 feet per year
Clayey Sand	1 to 10 feet per year

It is anticipated that the seepage of water around the proposed pump station will be small. Some seepage, however, will likely occur. The gradation of on site soils indicate that they are susceptible to piping during seepage.

Recommendations

(1) The compaction of backfill around the pump station is very important to controlling seepage along structure walls.
(2) The placement of the rip-rap filter in the channel downstream of the station will also act in the prevention of piping of fine soil particles.
(3) Use of cutoff walls will also assist in controlling seepage along the structure by increasing the length of seepage flow lines.

SEISMIC CONSIDERATIONS

General

The Uniform Building Code Seismic Zone Map of the United States places this site within seismic zone 3, fairly close to the boundary with seismic zone 4.

As referenced from the USGS Open File Report 82-1033 "Probablistic Estimates of Maximum Acceleration and Velocity in Rock in the Contiguous United States", by Algermissen and Perkins, et.al. 1982, seismologists have estimated that there is a 90% probability that this site will not experience horizontal ground acceleration from an earthquake greater than the following values in the time periods indicated.

Time Period	Acceleration
10 years	0.05 g
50 years	0.10 g
250 years	0.20 g

From these estimates, it can be extrapolated that for a 100 year design life the acceleration from an earthquake will be in the range of 0.14 g to 0.15 g.

Due to the high density of the in-situ soils it is anticipated that there is a very low potential for liquefaction at this site given earthquake shaking.

Recommendations

(1) The allowable soil bearing capacity can be increased by a factor of 1/3 under temporary seismic loading.
(2) For seismic evaluations of foundation and retaining walls, the equivalent fluid weight for the at rest and active condition should be increased by 25 pounds per cubic foot and the passive resistance should be reduced be 25 pounds per cubic foot.
(3) Facilities at the site should be constructed conforming to the regulations and standards presented in the uniform building code (UBC) for seismic zone 3 construction.

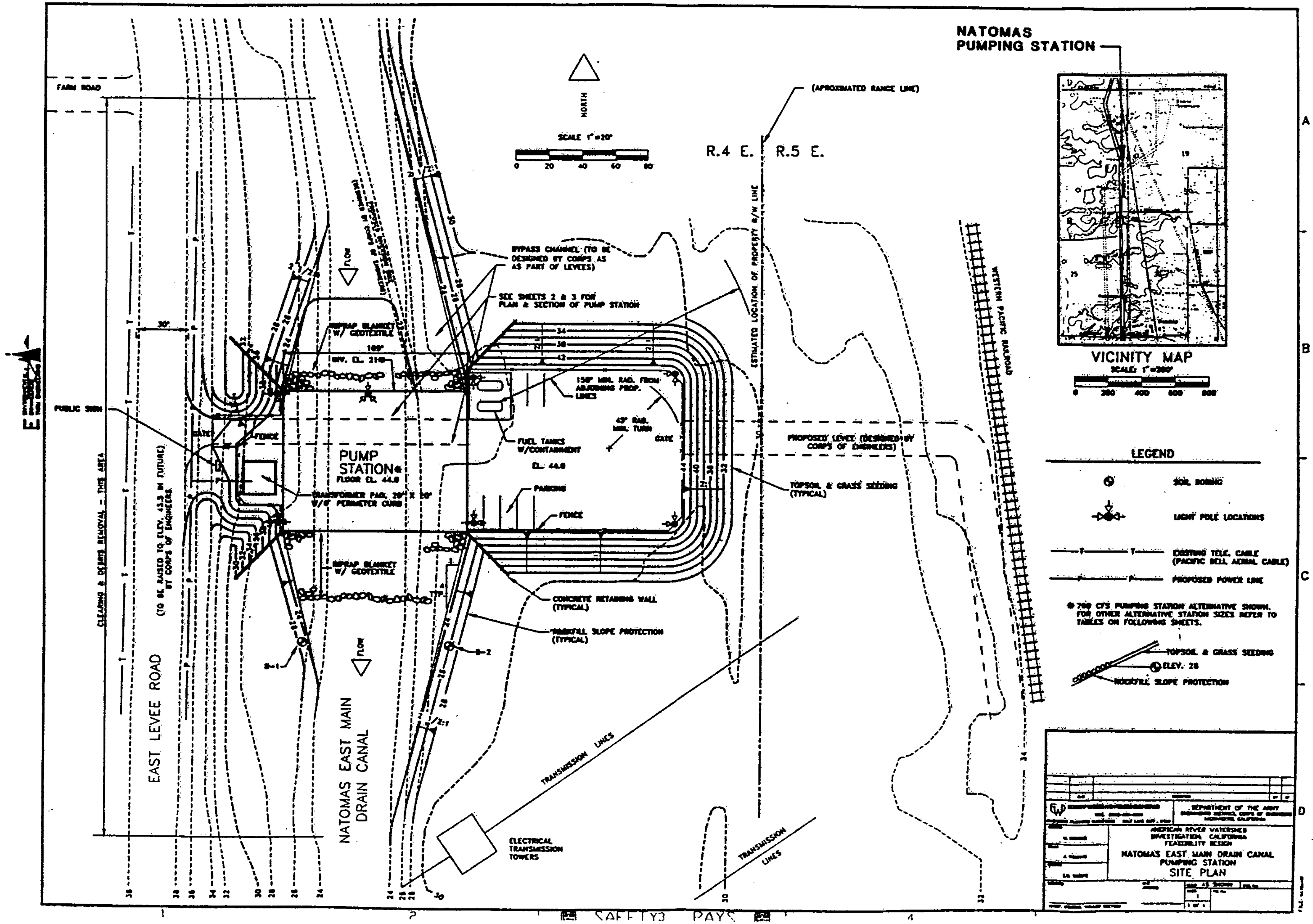
(4) Subsurface conditions at the site correspond most closely with type S-2 as referred to in the UBC. This designation should be used when determining the S factor from Table #23-J and the normalized response spectrum, Figure 3 in the UBC.

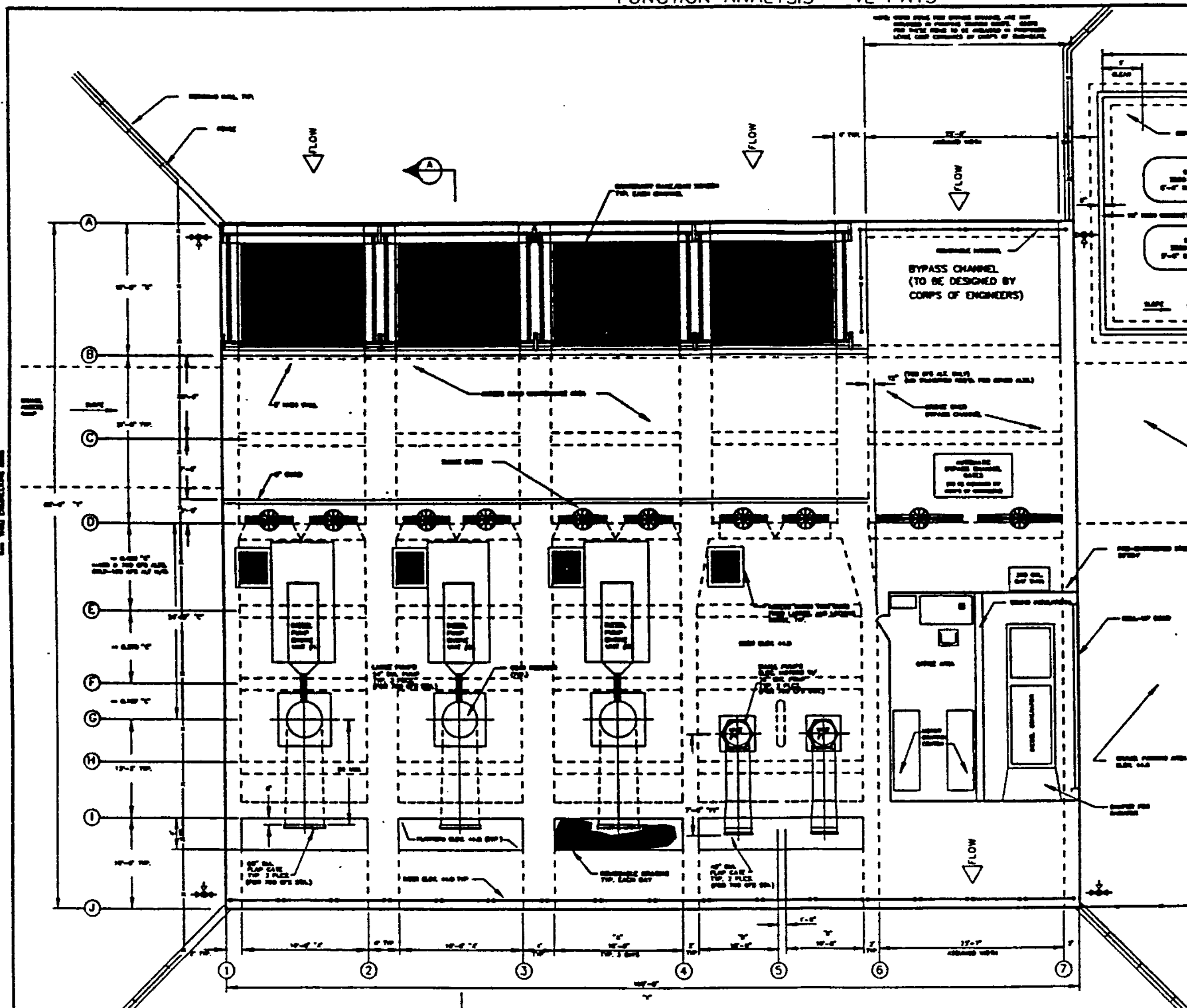
LIMITATIONS OF REPORT

This report has been prepared in accordance with generally accepted soil and foundation engineering practices in this area for use by the client for a feasibility design study. The conclusions and recommendations included in this report are based upon the data obtained from the borings drilled at the locations indicated on the site plan and the proposed site grading and construction as discussed in this report. The nature and extent of subsurface variations across the site may not become evident until final design investigations or excavations are performed.

APPENDICES A-E OF THE AE REPORT ARE NOT INCLUDED IN THE TECHNICAL REPORT

THESE APPENDICES ARE ON FILE IN THE CENTRAL VALLEY SECTION
OF THE CIVIL PROJECTS BRANCH
ENGINEERING DIVISION
SACRAMENTO DISTRICT


NATOMAS
PUMPING STATION
VICINITY MAP
SCALE: 1"=200'
LEGEND
SOIL BORING
LIGHT POLE LOCATIONS
EXISTING TELE. CABLE (PACIFIC BELL AERIAL CABLE)
PROPOSED POWER LINE
700 CFS PUMPING STATION ALTERNATIVE SHOWN. FOR OTHER ALTERNATIVE STATION SIZES REFER TO TABLES ON FOLLOWING SHEETS.
TOPSOIL & GRASS SEEDING
ELEV. 28
ROCKFILL SLOPE PROTECTION
NORTH
SCALE 1"=20'
R.4 E.
R.5 E.
(APROXIMATED RANGE LINE)
ESTIMATED LOCATION OF PROPERTY R/W LINE
WESTERN PACIFIC RAILROAD
FARM ROAD
PUBLIC 30th
BYPASS CHANNEL (TO BE DESIGNED BY CORPS AS AS PART OF LEVEES)
SEE SHEETS 2 & 3 FOR PLAN & SECTION OF PUMP STATION
RIPRAP BLANKET W/ GEOTEXTILE
PUMP
STATION
FLOOR EL. 44.0
TRANSFORMER PAD, 20" X 20" W/6" PERIMETER CURB
FUEL TANKS W/CONTAINMENT
EL. 44.0
PARKING
FENCE
GATE
150' MIN. RAD. FROM ADJOINING PROP. LINES
45' RAD. MIN. TURN
PROPOSED LEVEE (DESIGNED BY CORPS OF ENGINEERS)
TOPSOIL & GRASS SEEDING (TYPICAL)
CONCRETE RETAINING WALL (TYPICAL)
ROCKFILL SLOPE PROTECTION (TYPICAL)
FLOW
EAST LEVEE ROAD
(TO BE RAISED TO ELEV. 43.5 IN FUTURE) BY CORPS OF ENGINEERS
CLEARING & DEBRIS REMOVAL - THIS AREA
NATOMAS EAST MAIN DRAIN CANAL
ELECTRICAL TRANSMISSION TOWERS
TRANSMISSION LINES
DEPARTMENT OF THE ARMY
AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA FEASIBILITY DESIGN
NATOMAS EAST MAIN DRAIN CANAL PUMPING STATION
SITE PLAN



PLAN – 700 CFS PUMPING STATION

ALTERNATIVE PUMPING STATION DESIGNS

DESIGN FLOW REQUIRED* (CFS)	3 LARGE PUMPS		2 SMALL PUMPS		TOTAL PUMPING CAPACITY* (CFS)
	[illegible]	[illegible]	[illegible]	[illegible]	
100	30	24"	10	14"	110
400	120	42"	40	24"	440
700	210	54"	70	36"	770

GATES FOR ALTERNATIVE STATIONS

DESIGN FLOW REQUIRED (CFS)	[illegible]		[illegible]	
	[illegible]	[illegible]	[illegible]	[illegible]
100	4	4 X 4	30"	18"
400	4	6 X 6	48"	30"
700	8	5 X 6	60"	42"

NOTES:

1. WITH OUTAGE OF ONE SMALL PUMP, PUMPING CAPACITY IS 100% OF DESIGN FLOW. WITH OUTAGE OF BOTH SMALL PUMPS, PUMPING CAPACITY IS 90% OF DESIGN FLOW. WITH OUTAGE OF ONE SMALL AND ONE LARGE PUMP, PUMPING CAPACITY IS 70% OF DESIGN FLOW (ASSUMED WORST CASE).
2. LARGE PUMPS SIZED AT 30% OF DESIGN FLOW REQUIRED AND SMALL PUMPS AT 10%. TOTAL PUMPING CAPACITY IS 110% OF DESIGN FLOW.

PUMPING STATION DIMENSIONS (MIN.)

DESIGN FLOW REQ'D. (CFS)	OVERALL DIMENSIONS (FEET) X	OVERALL DIMENSIONS (FEET) Y	LARGE PUMP BAY(FT) WIDTH A	SMALL PUMP BAY(FT) WIDTH B	LARGE PUMP SUMP LENGTH (FEET) C	D	D₁	SMALL PUMP SUMP LENGTH (FEET) E	F	F₁	LENGTH OF CATENARY RAKE AREA (FEET) G
100	73	75'-5"	7	* 4	15	2'-6"	6'-2"	6	1'-9"	4'-9"	15'-8"
400	93	80'-5"	12	* 7	20	4'-0"	8'-5"	12	2'-6"	6'-2"	15'-8"
700	109	80'-9"	16	10	24	5'-4"	8'-9"	16	3'-6"	7'-8"	17'-0"

* MIN. WIDTH REQ'D. ACTUAL WIDTH IS 7.5' FOR 400 CFS ALT. & 5.0' FOR 100 CFS ALT.

DEPARTMENT OF THE ARMY

AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA FEASIBILITY DESIGN

NATOMAS EAST MAIN DRAIN CANAL PUMPING STATION

PUMP STATION PLAN

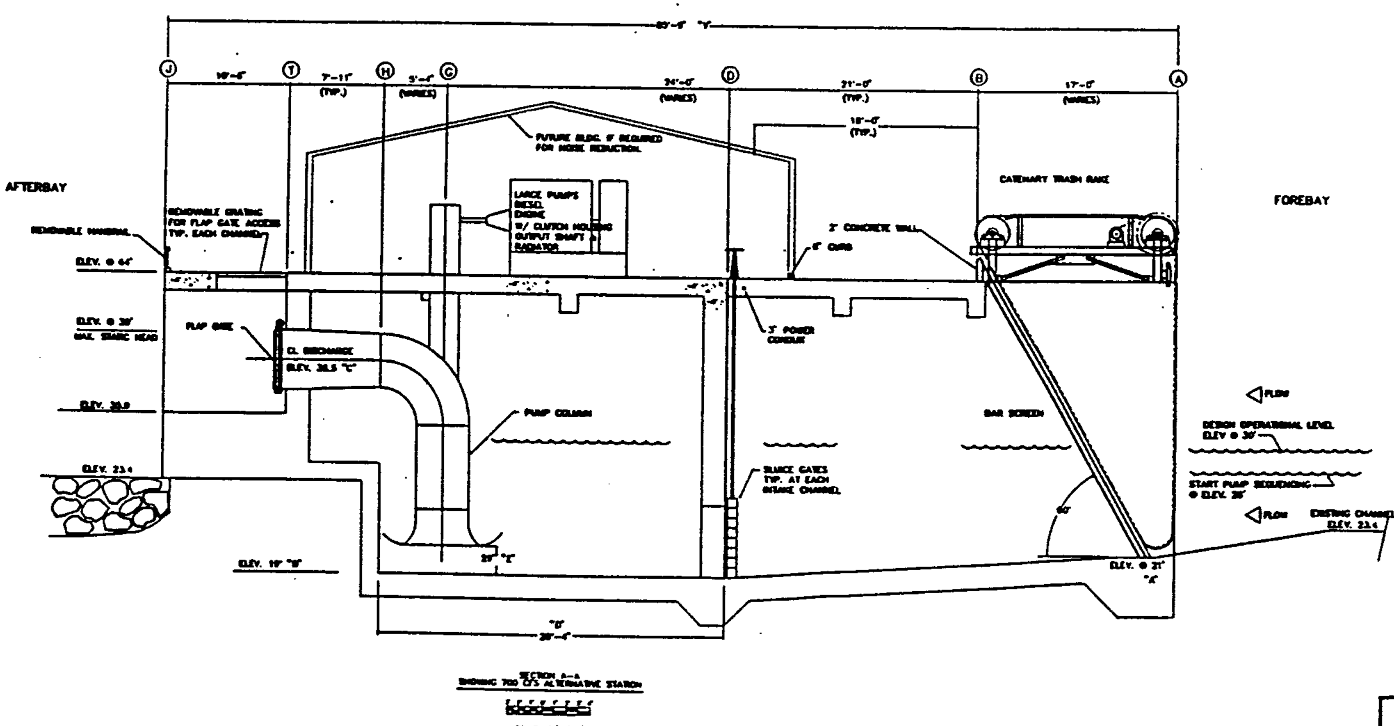
FUNCTION ANALYSIS – VE PAYS

PUMPING STATION ELEVATIONS

DESIGN FLOW (CFS)	"A" INVERT @ INTAKE	"B" ELEV. @ SUMP	ELEV. OF DISCHARGE "C" SMALL PUMPS	ELEV. OF DISCHARGE "C" LARGE PUMPS
100	23	20	38.25	37.75
400	23	20	37.75	37.00
700	21	19	37.25	36.5

PUMPING STATIONS LAYOUT

DESIGN FLOW (CFS)	"Y" LENGTH (FT)	"D" SUMP LENGTH (FT)	"C" LARGE PUMPS
100	73'-5"	17'-6"	17'
400	80'-5"	24'-0"	22'
700	85'-9"	29'-4"	29'



DEPARTMENT OF THE ARMY
AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA FEASIBILITY DESIGN
NATOMAS EAST MAIN DRAIN CANAL PUMPING STATION
LONGITUDINAL SECTION
SCALE AS SHOWN

SAFETY PAYS

LEGEND

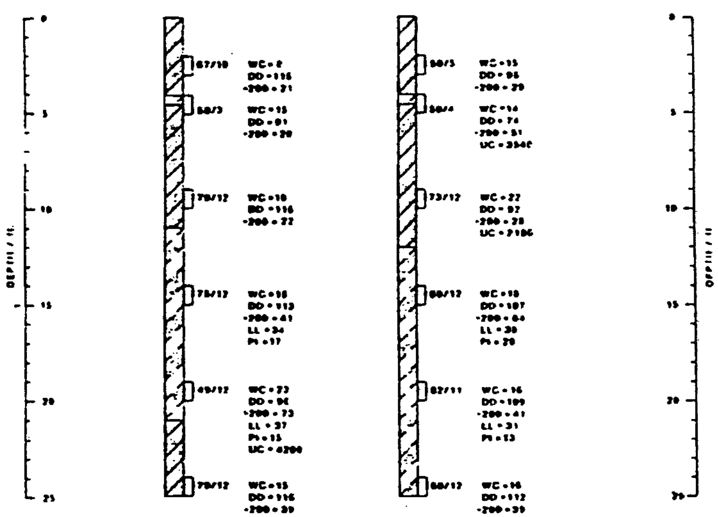
(SM) Sand silty, fine to medium grained, very dense. Slightly moist to moist, brown.

(SC - CL) Sand and clay, fine to medium grained, medium plasticity, very dense to hard. Moist, brown.

10/12 California Drive Sample. The symbol 10/12 indicates that 10 blows of a 140 pound hammer falling 30 inches were required to drive the sampler 12 inches.

B-1

B-2



NOTES

1. Borings were drilled on November 13, 1985 with a 7-inch diameter continuous flight hollow stem power auger.
2. Locations of borings were measured approximately by pacing from features shown on the site plan provided.
3. Elevations of borings were estimated from the contours indicated on the site plan provided.
4. The boring locations and elevations should be considered accurate only to the degree implied by the method used.
5. The lines between the materials shown on the boring logs represent the approximate boundaries between material types and the transitions may be gradual.
6. No water level observed. Fluctuations in the water level may occur with time.
7. WC = Water Content (%)
   DD = Dry Density (pcf).
   -200 = Percent Passing No. 200 Sieve.
   LL = Liquid Limit (%).
   PI = Plasticity Index (%)
   UC = Unconfined Compressive Strength (psf).

PREPARED BY CHEN NORTHERN, INC.
JOB No. 511598

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

NATOMAS EAST MAIN DRAIN
CANLL PUMPING STATION
BORING LOGS

S-2

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

APPENDIX N

CHAPTER 3

BASIS OF DESIGN AND COST ESTIMATES

DAM ALTERNATIVES

OCTOBER 1991

BASIS OF DESIGN AND COST ESTIMATES
DAM ALTERNATIVES

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont.)

LIST OF TABLES

TABLE OF CONTENTS (Cont.)

LIST OF TABLES

TABLE OF CONTENTS (Cont.)

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES

TABLE OF CONTENTS (Cont.)

LIST OF FIGURES

LIST APPENDICES

LIST OF PLATES

AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

CHAPTER 3

BASIS OF DESIGN AND COST ESTIMATES DAM ALTERNATIVES

GENERAL

This report will describe the design effort for the dam alternatives investigated and the selected plan. The site selection analysis for the dam is presented in Appendix J. River Mile (RM) 20.1 of the North Fork of the American River has been chosen as the site for all dam alternatives. This location is shown on Plate 1 and is approximately five miles southeast of Auburn, California. The dam and flood control pool would be in Placer and El Dorado Counties. This site is also the site of the United States Bureau of Reclamation's (USBR) authorized Auburn Project. Extensive analysis of this site by the Bureau, as well as actual construction, has been done. The Bureau's information and effort was used as much as possible in all designs and cost estimates.

The chapter first describes several initial analysis used to determine the type of dam to be used in designing the alternatives used in defining the National Economic Development (NED) plan. The chapter then goes on to describe the several alternatives. Costs developed for these alternatives are presented in this chapter. These initial alternative designs and costs are not M-CACES level designs or costs. They are designs and costs adequate to give enough information to be used for identification of the NED plan. After the NED plan was identified, this information along with other information was used to determine the tentatively selected plan. The selected plan was then analyzed using an M-CACES approach. This involved additional detailed design and a more thorough approach to the cost estimate using an M-CACES analysis. As analysis continued for the dam portion of the project, more and updated information was developed. Therefore, more detail was done for each additional step of the dam design. The dam alternatives analyzed for optimization of dam size were analyzed in more detail than the dams used for determining type of dam. The selected plan was analyzed in more detail than the dam alternatives. This increasing level of detail has sometimes resulted in changes in certain pertinent data which at first glance may appear to be inconsistent, but is due to the more detailed analysis done at each step in the formulation process. The additional analysis for the selected plan along with guidance from the Feasibility Resolution Conference, the Technical Resolution Conference, and the Project Guidance Memorandum resulted in design differences between the alternatives analyzed and the tentatively selected plan. The selected plan is described at the end of

the chapter. Differences between the alternatives analyzed and the selected plan are described there. The cost estimate for the selected plan is given in Chapter 4.

SITE CONDITIONS

The RM 20.1 site is in a deep river canyon with steep side slopes. The top of the canyon rim is approximately elevation 1300 feet with the bottom at elevation 470 feet. Width of the canyon is approximately 500 feet at the bottom and 3800 feet at the top. A significant fault was discovered in the left abutment during construction by USBR. Geology and seismicity at this site are discussed in more detail in Appendix M, Chapter 5. The USBR has done extensive construction at the site in connection with the authorized Auburn Project. A 2,350 foot long horseshoe shaped diversion tunnel in the left abutment has been constructed. Numerous access roads lead to the bottom of the canyon and to different elevations in the canyon. Foundation work for the authorized structure was almost complete before work was stopped. This includes extensive excavation and shaping as well as concrete dental work in the bottom of the canyon. A cofferdam was constructed upstream of the site which was breached during the 1986 floods. About one-third of the coffer dam was washed downstream over the RM 20.1 site. This cofferdam material covers some of the excavation work accomplished by USBR. The excavated keyway for the foundation is arch shaped, approximately 4,400 feet long, and is 500 feet wide at its widest point. The top of the excavation begins at about elevation 1100 feet.

MAPPING

Latest topography for the damsite and areas upstream and downstream was developed by USBR in 1979 and 1986. Maps are available with a 1" = 200' scale and contour interval of 5 feet which were used for design of the dam alternatives. Designs for the relocation of Highway 49 and Ponderosa Way were accomplished using 7.5 minute USGS quad sheets for the area. No surveyed profiles were available along the alignments for these relocations. During PED, updated maps will be prepared for the damsite. Cross sections as necessary will be taken across the river. Cross sections covering the expected right of way will be surveyed along the investigated alignments of the roadway relocations. Other mapping will be accomplished as necessary to determine land acquisition lines and requirements.

DAM ALIGNMENT

The axis of the dam was assumed to be straight for the early analysis of dam alternatives. The following constraints were considered in developing the dam alignment at RM 20.1: 1) locate the base of the dam off the major known fault in the left abutment as much as possible, 2) align the spillway with the canyon to prevent erosion of the canyon

walls during spillway flows, 3) stay out of the slide area downstream of the right abutment as much as possible, 4) retain use of the existing diversion tunnel, and 5) utilize existing foundation work as much as possible. Structural, Hydraulics, and Geotechnical personnel jointly developed an alignment that would best fit these constraints. Constraint number two was an important factor in determining the alignment of the dam because of the wide spillways required to pass the PMF event. The spillway widths are approximately as wide as the canyon bottom. It is considered undesirable to have spillway flows impacting the canyon walls. Plate 2 shows the alignment chosen for the RM 20.1 site for evaluation of dam alternatives. This plate also shows the trace of the major fault and the alignment the USBR has studied for its concrete gravity dam. Note that the USBR's preferred straight alignment for a multipurpose dam is different than the Corp's preferred alignment for a flood control only dam. This is due in large part to the difference in spillway requirements. This alignment is the alignment chosen for evaluation of different dam sizes and alternatives. Once the final plan is selected, the dam alignment will be further analyzed. This discussion is presented later in the report for the selected plan.

TYPE OF DAM

Two types of dam were evaluated at the site, rolled embankment and concrete gravity. A concrete gravity dam is considered to be the appropriate type of concrete dam to be evaluated based on the seismicity of the site, the proximity to an active fault, and previous work completed by the USBR. Currently, the roller compacted concrete (RCC) method of concrete placement is being widely used for concrete gravity dams and is considered to be the most economical method for a concrete dam at this site. RCC is described and discussed in Appendix M, Chapter 6. Chapter 6 also discusses the suitability of RCC for concrete dam alternatives at this site. While this would be one of the largest RCC dams in the United States, current technology and methodology is adequate for design and construction of an RCC dam of the sizes investigated at this site. The design would follow normal concrete design procedures using a very dry concrete mixture. Mixture and other tests will be conducted during the detailed design phase of any proposed project to insure that design concrete strengths and bond strengths will be realized. To determine which of these two types of dam would be least costly, preliminary designs and cost estimates were developed for single purpose 200-year flood control protection for each. Table N-3-1 gives data for the alternatives.

TABLE N-3-1

PERTINENT DATA - TYPE OF DAM ANALYSIS
RIVER MILE 20.1

Type of Dam	Level of Protection Years	Crest Elev. Feet	Max. Ht. of Dam Feet 1/	Crest Length Feet	Spillway Length Feet
RCC	200	934	434	2315	350
Rolled Embankment	200	941	441	2450	400

1/ Measured from streambed elevation of approximately 500 feet.

A rolled embankment design would be a zoned rockfill dam with cofferdams both upstream and downstream. Outlet works would consist of the existing horseshoe shaped diversion tunnel modified to a 30 foot diameter circular shape and a new parallel 12.5 foot diameter tunnel. The outlet works would be uncontrolled. A broadcrested spillway would be excavated in the left abutment. Material for the dam would come from the USBR borrow area 3000, from foundation and spillway excavation, and from the existing partial cofferdam.

The non-overflow section of the RCC dam would have a 30 foot wide crest, a vertical upstream face, and would have a 1V:0.75H downstream sloping face. A static compressive strength of 5,000 psi was used for design and quantity estimates. Small cofferdams will be required only during the initial stage of construction. The nonerosive nature of roller compacted concrete and the speed at which a RCC dam can be constructed do not require a large cofferdam to protect the construction area throughout the construction period. Outlet works would consist of the existing horseshoe shaped diversion tunnel modified to a 30 foot circular shape and two 5 foot wide by 9 foot high sluices through the dam. These outlet works would be uncontrolled. The spillway would be ogee shaped, constructed on top of the dam, and would terminate in a flip bucket. Table N-3-2 compares the cost for the two types of dam.

As can be seen, the concrete dam is less expensive. Also, a concrete dam is more easily expanded than a rolled embankment dam. This is an important consideration for this site, because one constraint, which has been described elsewhere in the report, is that a dam at this site should not preclude possible expansion in the future. Based on this cost analysis and the expandability constraint, it was decided that all remaining dam alternatives would be designed as roller compacted concrete gravity dams.

TABLE N-3-2

TYPE OF DAM COMPARISON
AT RM 20.1

ROLLED EMBANKMENT DAM			ROLLER COMPACTED CONCRETE DAM		
ITEM	DESCRIPTION	COSTS	ITEM	DESCRIPTION	COSTS
01	LANDS	26,800,000	01	LANDS	26,000,000
02	RELOCATIONS	83,000,000	02	RELOCATIONS	83,000,000
04	DAM	192,750,000	04	DAM	186,750,000
30	E&D	33,090,000	30	E&D	32,370,000
31	S&A	22,060,000	31	S&A	21,580,000
TOTAL PROJECT		357,700,000	TOTAL PROJECT		349,700,000

DAM ALTERNATIVES

Feasibility level designs and quantity estimates were developed for NED optimization for the proposed project. Different sizes of single purpose flood control dams were designed with each size also having a design that included future facilities. In addition to these flood control only alternatives, one design was developed for a dam which would provide a small permanent pool behind the dam. The total number of dams designed was seven. Freeboard was added to the probable maximum flood (PMF) pool elevations to determine the top elevation of the dams. Tables N-3-3 and N-3-4 provide pertinent data on size of dam and storages for the alternatives analyzed.

The hydraulic design of these dams was based on current hydrology, Appendix E, with a Probable Maximum Flood (PMF) peak inflow of approximately 1,068,000 cfs. All flood control only alternatives would use the existing diversion tunnel for flood control releases. The tunnel would be modified by lining with concrete and changing the shape to a 30-foot diameter circular section. Inlet and outlet structures would be added at each end of the tunnel. Additional flood control sluices would be provided through the dam. The number and size of these sluices would vary for flood protection provided. Two bypass sluices would also be provided to divert low flows during the summer while the flood control tunnel was inspected or maintained. The dams with future facilities, replace the flood control sluices with penstocks, which pass flood flows and which could be used for future power generation. More discussion on future facility options is provided later in this chapter.

TABLE N-3-3

ALTERNATIVE RCC DAMS FOR FEASIBILITY STUDIES
(RM 20.1)

Type of Dam	Level of Protection Years	Crest Elev. Feet	Max. Ht. of Dam Feet 1/	Crest Length Feet	Spillway Length Feet
Flood Control Only	100	851.5	401.5	2010	600
	200	933.0	483.0	2310	600
	400	1007.5	557.5	2540	540
Flood Control Only with Future Facilities	100	851.5	401.5	2010	600
	200	933.0	483.0	2310	600
	400	1007.5	557.5	2540	540
Expandable Minimum Pool	200	968.7	518.7	2390	600

1/ Measured from foundation elevation of approximately 450 feet.

None of the outlet works for the flood control only alternatives are controlled by gates. Bulkhead gates are provided to block the passageways during inspection and maintenance. All alternatives include an ogee shaped spillway on top of the dam. The minimum pool alternative includes gated sluices for release of low flows and emergency drawdown. Table N-3-5 gives details of the outlet works for the dam alternatives. Preliminary structural design of the non-overflow section of the dams was based on seismic stress analyses. Spillway and outlet works structural design and quantities were based on experience and previous projects and estimates. Foundation quantities and dental work were derived from the USBR's Auburn Dam Interim Construction Report [3] (numbers in brackets refer to references noted at the end of this Appendix) which describes the efforts required for the foundation work for the authorized Auburn Project.

TABLE N-3-4

DAM ALTERNATIVES
RESERVOIR STORAGE INFORMATION

PURPOSE	FC POOL FREQ	PERM POOL ELEV (FT NGVD)	PERM POOL STORAGE (AC-FT)	FC POOL ELEV (FT NGVD)	FC POOL STORAGE (AC-FT)	TOTAL CONTROLLED STORAGE (AC-FT)	MAX POOL ELEV (FT NGVD)	MAX POOL STORAGE (AC-FT)
FC ONLY	100-YEAR	NA	NA	784	270,000	270,000	844.5	454,000
FC ONLY	200-YEAR	NA	NA	869	545,000	545,000	926.0	807,000
FC ONLY	400-YEAR	NA	NA	942	894,000	894,000	1,000.5	1,245,000
NOTE: INFORMATION FOR DAMS WITH FUTURE FACILITIES IS SAME AS ABOVE								
FC W/ MIN POOL	200-YEAR	715	127,000	905.5	579,000	706,000	961.7	1,003,000

NOTE: ALL ABOVE INFORMATION BASED ON 400,000 AC-FT FC STORAGE IN FOLSOM RESERVOIR

TABLE N-3-5

DAM ALTERNATIVES
OUTLET WORKS DESCRIPTION

Type of Dam	Level of Protection Years	Outlet Works Provided
Flood Control Only	100	30-Foot Tunnel, 10 - 5'W X 9'H Sluices
	200	30-Foot Tunnel, 2 - 5'W X 9'H Sluices
	400	30-Foot Tunnel with Restricted Opening, 1 - 5'W X 9'H Sluice
Flood Control Only with Future Facilities	100	30-Foot Tunnel, 2 - 18'D Penstocks with terminal restrictions
	200	30-Foot Tunnel, 2 - 18'D Penstocks with 1 Sealed and 1 with terminal restriction
	400	30-Foot Tunnel with Restricted Opening, 2 - 18'D Penstocks with 1 Sealed and 1 with terminal restriction
Minimum Pool with Future Facilities	200	3 - 5'W X 9'H Sluices with Bulkheads, Service, and Emergency Gates. These Sluices are Required for Emergency Drawdown Procedures. 6 - 10'W X 21.4'H Ungated Flood Control Sluices 2 - 18' D Penstocks, Both Sealed

FUTURE FACILITY OPTIONS

Two primary differences exist between the dam alternatives which do and do not have future facilities. (1) The dams with future facilities include penstocks and intake structures for future use. These steel penstocks are 18 feet in diameter and were sized to accommodate an approximately 300-400 Megawatt future power plant. In some cases the penstocks are used for flood control releases and in others the penstocks would be provided in the dam and sealed until need for them occurs. (2) The cross section geometry of the non-overflow and overflow sections is different. Plate 3 shows typical cross sections of the non-overflow sections of the dam alternatives. The continuous sloping downstream face for the expandable dam is necessary to ensure a strong bond and proper load transfer between old and new concrete if the dam is ever expanded. Plate 4 shows typical spillway sections. If the dams were expanded, the spillway would have to be covered and a new spillway constructed for the expanded structure. Plate 5 shows an elevation of the downstream face of the alternative dams and Plate 6 shows approximate plan views of the 100 year, 200 year, and 400 year levels of protection dams.

RELOCATIONS

The major relocation required for any of the dam alternatives is California State Highway 49 (Highway 49). This highway connects Auburn and Placerville, California and is the major route for people travelling between these two towns and the foothill area between them. Other relocations include power and telephone lines which cross the river along the alignment of the existing Highway 49 and a county road (Ponderosa Way) crossing in the upper reaches of the North Fork of the American River. More detailed discussion of the design of the relocations follows.

California State Highway 49

California Department of Transportation (CALTRANS) criteria were used to design the Highway 49 relocation for the following reasons:

1. The owners criteria should be used whenever possible.
2. Should this project be constructed, CALTRANS will most probably design and construct Highway 49.

The proposed Highway 49 has been relocated as close as possible to the current alignment and is designed to match the existing roadway. The road has a design speed of 55 miles per hour. Design criteria are minimum 1500 foot horizontal curve radius, minimum 6% grades, and vertical curves based on sight distance. The road will be two lane as is the current highway.

The objective was to place the relocated highway as low as possible

in the canyon at an elevation just above the flood control effects of the project. The relocation is approximately 9,300 feet in length and consists of four bridges and a short span of connecting roadway. The bridges are a total of 8,900 feet and the roadway is 400 feet. The road profile was kept above elevation 995, maximum pool elevation for the proposed project, at all locations. The bridges cross the North Fork of the American River and several ravines which drain into the American River. This relocation exits and rejoins the existing Highway 49 alignment in the American River Canyon. It replaces 13,000 feet of canyon roadway with 9,300 feet of bridge and roadway. Since the alignment is essentially all bridge and viaducts, it is an expensive relocation. The alignment is shown on Figure 1 and the costs are given in Table N-3-6.

This relocation is considered to be in-kind for the existing Highway 49 and is all that is required to accommodate the Federal flood control project. The local sponsor or CALTRANS may wish a different alignment or wider roadway. These improvements are considered to be betterments and all studies and costs for these would be borne 100% by the non-Federal sponsor.

TABLE N-3-6

HIGHWAY 49 RELOCATION COSTS
(RM 23.0)

LANDS	$ 1,100,000
BRIDGES	$ 72,300,000
TOTAL	$ 73,400,000

Ponderosa Way

Another road, Ponderosa Way crosses the North Fork of the American River. The existing road is winding, steep, and has a dirt surface. The road is currently used for mining, water recreation, access to homes, fire protection, and transportation between foothill communities.

Ponderosa Way has been designed as a one way road at the location where a new bridge is required to replace the existing bridge. No requirement was placed on the horizontal alignment. The vertical alignment was held below 14 percent with the bridge crossings at less than 2 percent. The road will be 12 foot wide with 2 foot shoulders. The road structural section will have a 2 inch bituminous surface and a 6 inch base course. In cut the shoulder will be paved to serve as a side ditch, and in fill the shoulder will be base rock. Both CALTRANS criteria and Corps of Engineers criteria agree to the parameters of the structural section for this road. Although the roadway is only one lane, the bridge structure has been designed for two lanes for safety reasons. The bridge structure is the major portion of the relocation.

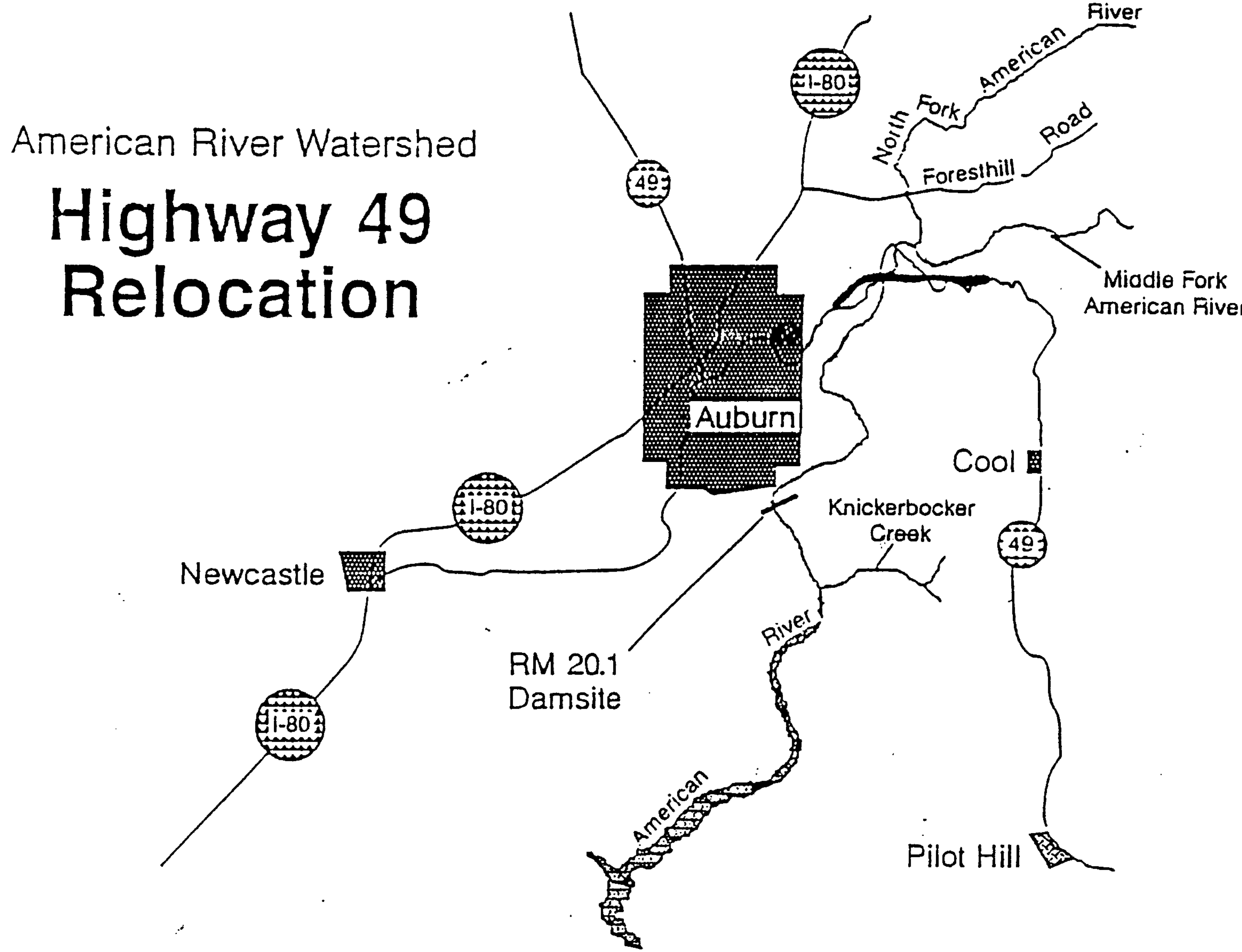

American River Watershed
Highway 49
Relocation
I-80
49
River
American
Fork
North
Road
Foresthill
Middle Fork
American River
Auburn
Cool
I-80
Knickerbocker
Creek
49
Newcastle
RM 20.1
Damsite
River
I-80
American
Pilot Hill

FIGURE 1

Power Lines and Telephone Lines

Both power lines and telephone lines will be relocated from existing lines to Cool or Highway 49 in El Dorado County. Existing lines are located at the ends of the alternative alignment to be reconnected to the relocated utility lines. Both power lines and telephone lines will be located in the road right-of-way. When these lines cross the American River, they will be located in race ways constructed in the bridge. Both power lines and telephone lines will be placed underground, as required by the Public Utilities Commission (PUC).

The existing underground power lines and telephone lines that are located in the flood control pool or are no longer required or needed will be abandoned. The overhead utilities will have a salvage value for +he wire and probably for the poles. The underground utilities have been considered to have no salvage value.

MISCELLANEOUS REQUIREMENTS

Clearing

For the flood control only dams no clearing is required. For the dam alternative which provides a minimum pool, the area within the permanent pool would be cleared, approximately 1,800 acres.

Project Roads

Access roads to the top of dam and outlet works are required and a design for these roads has been included in the investigation. The Top of Dam Road has three alternatives to account for the three elevations of the proposed dam. No control was placed on the horizontal alignment and a 15 percent limit has been placed on the vertical alignment of the access roads to the top of dam and to the outlet works. For the access roads to the outlet works a 20 percent limit has been used for the vertical alignment.

Portions of the access roads to the top of dam and to the outlet works have been located on existing roads. For these sections of road a 18 foot wide, 2 inch thick bituminous blanket is proposed. At locations where new roads have to be constructed, the road has been designed with two 9 foot lanes and 3 foot shoulders. In cut, the shoulder will be paved and has a bituminous dike to serve as a side ditch. In fill, the shoulder will be base rock. The structural section will be a 2 inch bituminous surface course over a 6 inch base course. Drainage and safety facilities are included in the design of these roads.

Portions of the access road to the outlet works will not be paved but will have a 6 inch rock surface. The road will be 20 feet wide. This road has been designed for track driven cranes. This road will have both a side ditch and a top of cut ditch to catch and convey runoff. Other drainage facilities as well as safety facilities are

included in the design of this road.

Recreation Facilities

For the alternative which provides a minimum pool, minimum recreation facilities will be located at the ends of existing roads and other places that will invite the public to be near the water. These facilities will be the minimum required to serve the public health and safety. Each area consists of a 30 car paved parking area, 3 chemical toilets, and 4 trash receptacles.

REAL ESTATE REQUIREMENTS

Real Estate requirements for the dam alternatives were developed using the criteria in ER 405-1-12. These land requirements were provided to Sacramento District Real Estate Appraisers who developed a gross estimate for the value of the required lands. This estimate is described in the Real Estate Division Report Number 89-068. The following are excerpts from this report.

Real Estate gross estimates for the value of lands under three dam alternatives along the American River were developed. These alternatives are:

1.--Flood control only dam with no future provisions for permanent storage (flowage easement value). This alternative is based on 100-year, 200-year, and 400-year floods, with pool elevations of 784 feet, 869 feet, and 942 feet respectively.

2.--Flood control dam with future facilities which would not preclude future expansion, includes all lands required for the USBR authorized 2.3 million acre-foot reservoir (fee value).

3.--Primarily flood control dam but with a minimum pool primarily for incidental local water supply (fee value). The minimum pool dam is at elevation 715 feet and the flood control elevation has been set at 920 feet for a 200-year flood.

Also provided is an estimate of the fair market value of the lands outside the required take areas under the 3 scenarios of Alternative 1 but still within the USBR's 2.3 million acre-foot multiple purpose take area required for the authorized Auburn Project.

The following assumptions were used in developing the real estate estimates:

a. Information is believed to be true and factual by the appraiser, but no responsibility is assumed for information supplied by others.

b. It is assumed that the title of all properties in the proposed take area is free and marketable.

c. The value estimate for Alternative 1 is based on flowage easement values except for the damsite, access road and administration area which is valued at fee. The value estimate for Alternative 2 is based on fee acquisition, and the value estimate for Alternative 3 is also based on fee acquisition values.

d. Comparable data information used to estimate the subject's values are in the field folder maintained in the Sacramento District's Real Estate Division Appraisal Branch.

e. The improvements values were based on typical cost of structures and not on individual evaluations. The number and type of improvements are based on information on the Assessor's roles of each county.

f. No toxic wastes or contaminations were observed during the inspection of the project area. However, it is felt that a thorough investigation should be completed; to determine whether in fact, there are any toxics or contaminates present. This thorough investigation will be done during PED.

The proposed project will involve the application of Public Law 91-646 and Title VI of Public Law 100-17 in the relocation of persons and personal property. These costs have been listed under acquisition cost in the estimates. Real Estate acquisition costs are those expenditures required by the various disciplines involved in the acquisition process by the local sponsor (Appraisal, Mapping, Surveying, Title Evidence, Negotiating, Closing and Condemnation). These costs have been estimated at $15,000 per private ownership, and $7,000 per governmental agency (USBR, BLM, State of California, etc.). These costs are also included in the acquisition costs in the estimates.

The contingency percentage used includes severance damage estimates, property appreciation, minor project design changes, undetected improvements, restricted access for inspection, unknown property splits, and any additional costs associated with the application of PL 91-646 and PL 100-17. The contingency percentage applied is 15 percent.

No commercial mineral operations were found during the inspection of the property. Therefore, it has been assumed that no marketable minerals are located in the subject area.

All publicly owned lands (Federal, State, and County) have been valued at their fair market value. Under Alternative 1, the rights to be acquired are flowage easement rights (occasional flooding) which have been calculated at 75% of fee. However, there are an estimated 565 acres which will encompass the damsite, access roads, and administration areas which will be acquired in fee. Under Alternatives 2 and 3 , the rights to be acquired are all in fee.

The subject area was physically inspected by an appraiser during the months of May and June, 1989. The subject area is located within El Dorado and Placer Counties along the North and Middle Forks of the American River. Land uses within the area consists of single-family residential lots (improved and unimproved), rural residential parcels (improved and unimproved), forest, recreation, commercial, and vacant rural lands.

Alternative 1 has three elevations. The dam with the 784 foot gross pool elevation provides a 100-year level of protection and covers approximately 9,100 acres, the 869 foot gross pool elevation dam provides a 200-year level of protection and covers approximately 18,800 acres, and the 942 foot gross pool elevation dam provides a 400-year

level of protection and covers about 21,100 acres. The USBR has previously acquired 6,610 acres under the 100-year dam, about 13,770 acres under the 200-year dam, and about 15,400 acres under the 400-year dam.

Alternative 2 is for a flood control dam with provisions to not preclude expansion. The ultimate possible land requirement under this alternative is approximately 42,120 acres, of which some 28,010 acres, were previously acquired by the USBR.

Alternative 3 is for a minimum pool reservoir (at elevation 715 feet) with flood control provisions to elevation 920 feet which will provide 200-year protection. This alternative covers approximately 20,100 acres. The USBR has previously acquired 14,450 acres under this alternative. The land within the minimum pool take area and the land under the flood control area are to be acquired in fee. The take area acre figures for Alternatives 1 and 3 were developed by the Corps of Engineers, while the USBR determined the acres for the maximum 2.3 million acre-foot reservoir in Alternative 2.

Highest and best use is defined as that legal use which will provide the greatest net return to the land over a reasonable period of time. The current land uses in the subject area appear to be their highest and best uses. Residential, rural residential homesites, and commercial lands appear to be increasing somewhat in value, while the forest, recreation, and rural lands appear to be maintaining their present values. There have been very few sales outside the populated areas within the project area, while sales near the populated area have been taking place on a steady basis. The following lists the value ranges within the project area:

Type of Land	Value Range
EL DORADO COUNTY	
Residential	\$10,000 - \$ 35,000 per lot
Rural Residential	\$ 4,000 - \$ 25,000 per acre
Commercial	\$20,000 - \$100,000 per acre
Rural	\$ 1,000 - \$ 2,000 per acre
Recreational	\$ 5,000 - \$ 10,000 per acre
Forest	\$ 250 - \$ 1,000 per acre

Type of Land	Value Range
PLACER COUNTY	
Residential	\$20,000 - \$ 50,000 per lot
Rural Residential	\$ 4,000 - \$ 25,000 per acre
Commercial	\$50,000 - \$150,000 per acre
Rural	\$ 2,000 - \$ 10,000 per acre
Recreational	\$ 250 - \$ 15,000 per acre
Forest	\$ 500 - \$ 1,000 per acre

Table N-3-7 gives the real estate cost estimates developed for the different dam alternatives.

TABLE N-3-7
DAM ALTERNATIVES
REAL ESTATE COST ESTIMATES
OCTOBER 1989 PRICE LEVEL

ALTERNATIVE 1	
100-YEAR FLOOD CONTROL ONLY DAM	$20,000,000
200-YEAR FLOOD CONTROL ONLY DAM	$24,000,000
400-YEAR FLOOD CONTROL ONLY DAM	$29,000,000
ALTERNATIVE 2	
LANDS REQUIRED FOR 2.3 MILLION ACRE-FOOT RESERVOIR	$73,000,000
ALTERNATIVE 3	
MINIMUM POOL FLOOD CONTROL DAM	$32,000,000

OUTLET WORKS AND SPILLWAYS

The following information presents a basis of hydraulic design for sizing the spillway and outlet works facilities for the dam alternatives. It includes design criteria, methodologies, and typical component configurations for the spillway and outlet works. All hydrologic data were obtained from the recently developed Hydrology Report, Appendix K.

Spillway Design

Spillway crest elevations are set such that only flows in excess of the design level of protection will pass over the spillway. The crest elevations were set at the flood control pool elevations shown in Table N-3-4. The spillway crest was designed as an ungated standard ogee with an elliptical upstream quadrant and a parabolic downstream quadrant. The downstream quadrant becomes tangent to the spillway chute at a slope of 0.7H on 1.0V. A flip bucket will be used to dissipate energy clear of the downstream toe of the dam. The flip bucket invert is located above the PMF tailwater in order to insure proper functioning. See Plate 4 for a typical profile of the spillway.

The selection of the Spillway Design Flood (SDF) was made in accordance with EC 1110-2-27, 'Policies and Procedures Pertaining to

Determination of Spillway Capacities and Freeboard Allowances for Dams.' Technical design of the spillway and appurtenances was made in accordance to EM 1110-2-1603, "Hydraulic Design of Spillways," and HDC 100-Spillways.

Spillway crest lengths were selected to minimize dam height while still allowing the jet from the spillway flip bucket to fit in the canyon downstream. The spillway rating curve was computed by assuming a high ogee weir condition and computing discharge coefficients as described in HDC, Sheet 111-3.

The Probable Maximum Flood (PMF) was selected as the SDF according to the definition of the functional design Standard 1 presented in EC 1110-2-27. The PMF hydrograph was routed through the reservoir assuming no outlet works release and an initial pool at spillway crest elevation. The resulting maximum pool elevations are given in Table N-3-4. The routing method is based on the principle of continuity, commonly called the modified Puls routing method. This method equates the difference in the average inflow and outflow over a specific time period (dt) to the change in storage. For the SDF routing, a time period of one hour was used.

Energy dissipation for spillway flow would be accomplished by using a flip bucket. A flip bucket was chosen because of a) the good quality of rock downstream and b) the difficulty in using a roller bucket configuration due to tailwater conditions. The flip bucket was designed using criteria presented in EM 1110-2-1603, "Hydraulic Design of Spillways". The tailwater elevation was computed using the program HEC-2, "Water Surface Profiles."

Outlet Works Design

The outlet works include an existing tunnel through the left abutment and sluices through the dam itself. The existing diversion tunnel (after modification) is used to discharge as much of the design flood flows as possible, while the sluices are required to pass remaining flow. These sluices have trash struts, intakes suitable for high head operation, and flip buckets at the outlets to dissipate energy well downstream from the toe of the dam. No bulkheads are required on the flood control sluices, since the inverts are set just above the pool elevation required to pass the summer low flows through either the diversion tunnel or a set of tunnel bypass sluices. The tunnel bypass sluices serve only to pass the low summer flows of 2100 cfs during periods of tunnel inspection or maintenance. The bypass sluices measure 5'W x 9'H, are bulkheaded, have short radius entrances, and no trash struts. The bypass sluices must be bulkheaded off during flood flows. The intake elevation of the tunnel will remain at 491.5 feet. The tunnel intake will consist of dual portals with elliptical roof and side curves transitioning to a 30 foot diameter conduit. A trashrack will be provided at the intake entrance.

Discharge Capacity

The primary outlet discharge capacity was calculated to control the design flood to an acceptable level for Folsom Dam and Reservoir and the lower American River system. Drawdown criteria are not a controlling factor for outlet works releases. The technical design criteria presented in EM 1110-2-1602, "Hydraulic Design of Reservoir Outlet Works", and HDC 200 - Outlet Works, were used for sizing and rating of the outlet works. Diversion during construction will be through the existing diversion tunnel.

The existing 33 foot horseshoe tunnel will be modified and utilized. This tunnel was designed for diversion during construction and was not meant to handle high head pressure flows. Therefore, the horseshoe shape will be lined with concrete to obtain a 30 foot diameter circular section which will carry these high head flood release flows. The sluices were sized as rectangular conduits. This cross section shape allows for easy maintenance and also allows full boundary layer development before the water exits the sluice. Different numbers of sluices were necessary, in addition to the tunnel, to pass the different flood control flows. The number and size of sluices were presented in Table N-3-5. The maximum discharge and outlet works ratings were computed using the following equation:

$$Q = \left[\frac{2g(PE-EI-Yp)}{\text{Sum K}} \right]^{1/2} A$$

Where:

Q = Discharge (cfs)	PE = Pool Elevation (feet)
Yp = Pressure Gradient at Exit	EI = Exit Invert Elevation (feet)
A = Area of Conduit (sq ft)	Sum K = Summation of Loss Coefficients
g = Acceleration of Gravity	

The summation of loss coefficients (Sum K) were limited to intake losses, conduit friction losses, and exit losses. All loss coefficients were converted in relation to the conduit area.

The rating curves developed for the tunnel and sluices were used to perform the reservoir routings. Routings consisted of an analysis of the American River Basin including operation and storage requirements for Folsom Dam and Reservoir. These routings established the storages required to control the desired frequency floods.

Tunnel Entrance Structure

An entrance structure will be added to the existing tunnel and consist of trash control facilities and bulkhead. Trash control will be accomplished by a lograck fabricated of horizontal and vertical members of corrosion resistant tubular steel. Clear openings between the lograck members will be 10 feet horizontally and 20 feet vertically.

The dimensions of the lograck are determined using criteria established in EM1110-2-1602, Paragraph 3-5b. Two bulkhead gates (one in each entrance portal) will be used for emergency and inspection closure of the tunnel. The bulkhead gates will be operated by hydraulic cylinders mounted atop the intake structure. Bulkhead slots will be provided in the intake structure walls to guide the gates into place. The gates will measure 15 feet wide by 39 feet high. The intake structure will have an entrance shape consisting of converging wall and roof curves conforming to criteria set forth in EM 1110-2-1602, Paragraph 3-6. Both wall and roof curves will converge to a transitional shape 15 feet wide by 30 feet high in each of the two entrance portals. These sections will converge to form the 30 foot diameter circular tunnel in accordance with EM 1110-2-1602, Paragraph 4-22c and Appendix D.

Sluice Intakes

The flood control sluice intakes will not be bulkheaded but will have trash racks and elliptical intake curves for both roof and walls. The tunnel bypass sluice intakes will be bulkheaded, have no trash control measures, and have short radius intake curves.

Energy Dissipation

Flip buckets will be used to dissipate energy downstream of the outlet works. The existing tunnel exit will be modified to attach a flip bucket. The high level sluices will also have flip buckets at their exits. The low level sluices will not have any energy dissipaters as they will only be used intermittently to pass low flows. In all cases, the tailwater will act as a plunge pool.

No protection is planned downstream at this time as the quality of rock found at the site appears to be adequate to withstand erosion from flood flows.

STRUCTURAL ANALYSIS

The following important assumptions were made for the feasibility designs of the alternative dams: 1) Stresses, particularly those due to seismic response in the dam, control the cross section geometry rather than overall stability of the dam. It is felt that if the dam can withstand the stresses due to the seismic load, it will be stable for all other loads. 2) Foundation work is similar and proportional to the work completed for the USBR's arch dam. 3) Any future expanded multipurpose dam could be constructed to approximately the same height, elevation 1142 feet, as the USBR's authorized multipurpose dam.

Preliminary seismic stress analyses were performed to aid in the feasibility level designs. This report provides details of the analyses, evaluates, and reports the results of this analysis later in this report. The primary source of data and information for the seismic

studies was the USBR with references [5] and [6] being particularly useful. The USBR provided the Corps with the synthetic earthquake accelerograms that were used to model the Maximum Credible Earthquake (MCE) for the RM 20.1 Site. Briefly, the MCE can be described as a magnitude 6.5 earthquake with a peak ground acceleration of 0.64g in the horizontal direction and 0.39g in the vertical direction. Additionally, the USBR was required to design for a fault displacement of nine inches. The MCE is considered to be very conservative by both the USBR and the Corps, however, there was no attempt to relax the current requirements for these dam alternative analysis.

Seismic analyses were limited to the 200 year level of protection dams and the multipurpose dam at the RM 20.1 site. Parametric studies were also very limited. The multipurpose dam was analyzed to determine if this size of dam was viable and also to evaluate the stresses in the expandable section after any possible future expansion. Fault displacement studies were not done. However, the USBR's seismic analysis of their proposed CG-3 dam, reference [5], indicates that a nine inch fault displacement does not produce adverse stresses in a concrete gravity dam at this site. Also, providing closely spaced joints in the dam is considered to be a better way of handling fault displacements, as opposed to designing for stresses. Results of the studies indicate that a flood control only dam and/or a multipurpose dam could safely be constructed at the RM 20.1 site.

Computer Program

Two-dimensional finite element analyses of three alternatives for the dam were performed with the use of the computer program EAGD-84. This program is designed specifically for two-dimensional earthquake analysis of concrete gravity dams [7,8,9]. The program was developed at the University of California at Berkeley by Dr. Gregory Fenves and Professor Anil K. Chopra. A detailed description of input data for the program is available in reference [10].

Basically, EAGD-84 performs a substructure analysis in the frequency domain and computes the response of a concrete gravity dam subjected to an arbitrary earthquake ground motion. The simultaneous effects of dam-reservoir interaction, dam-foundation rock interaction, and reservoir bottom wave absorption are included in the analysis.

Dam Finite Element Model

During large-amplitude earthquake vibration, the inertia forces developed are much greater than the shear forces that can be transmitted across joints between the dam monoliths. Consequently, the joints would slip and the monoliths tend to vibrate independently. Considering the effects of the joints, a two-dimensional plane stress model of the individual monoliths is appropriate for predicting the earthquake response. The tallest, non-overflow monolith for each of the alternatives analyzed was selected for the finite element analysis.

The dam monolith is idealized as an assemblage of planar, four-node non-conforming finite elements connected at the nodal points in the global X-Y plane. The X-axis is horizontal and positively directed downstream; the Y-axis is vertical with the positive direction upwards. The nodes at the base of the dam must be equally spaced in order to consider the effects of dam-foundation rock interaction. The finite element mesh should be laid out in such a way that element aspect ratios should not be excessive. They should be on the order of 1:1, and preferably less than 4:1. Elements in the shape of parallelograms with aspect ratio near unity usually give the most accurate analysis results. The smallest bandwidth of the structural stiffness matrix, and hence lower computational cost can be achieved by numbering the nodal points in the direction of the monolith cross-section with the smallest number of elements. However, the numbering of elements can be arbitrary because it has no effect on the computational effort.

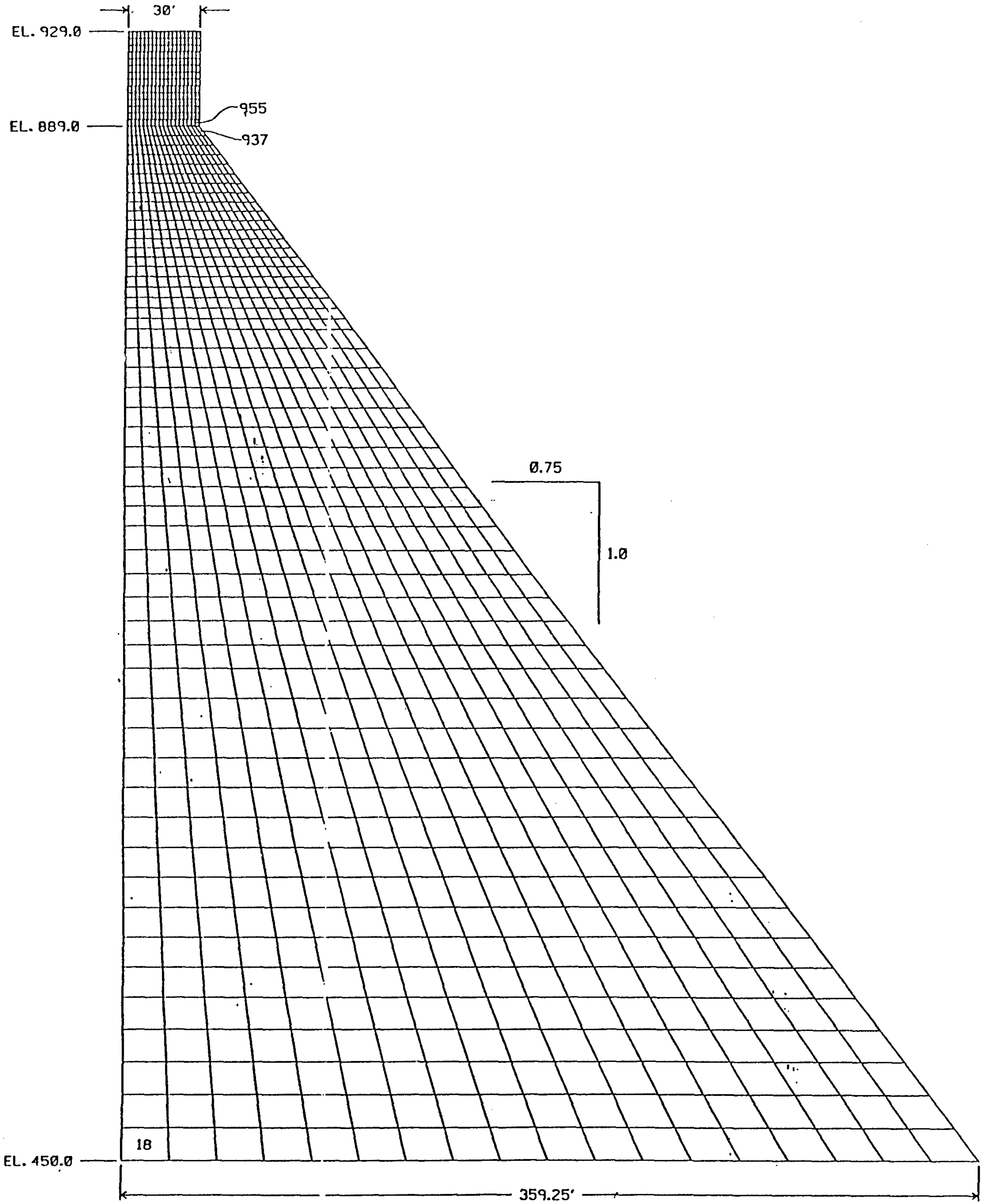
The finite element models for the flood control only dam, dam with future facilities, and multipurpose dam are shown in Figures 2, 3, and 4, respectively. These dam meshes were determined to be sufficiently refined to give accurate stress results. Table N-3-8 contains a summary of the features of the finite element idealization. Note that two translational degrees of freedom (DOF) are associated with each free node. Considering foundation-rock flexibility, the nodes at the base of the dam are free to translate in the X, Y-directions.

TABLE N-3-8
FEATURES OF DAM MESHES FOR EAGD-84 ANALYSIS

	Number of Nodes	Number of Elements	Number of DOF
Non-expandable dam	1292	1206	2584
Expandable dam	1349	1260	2698
Multipurpose dam	2208	2093	4416

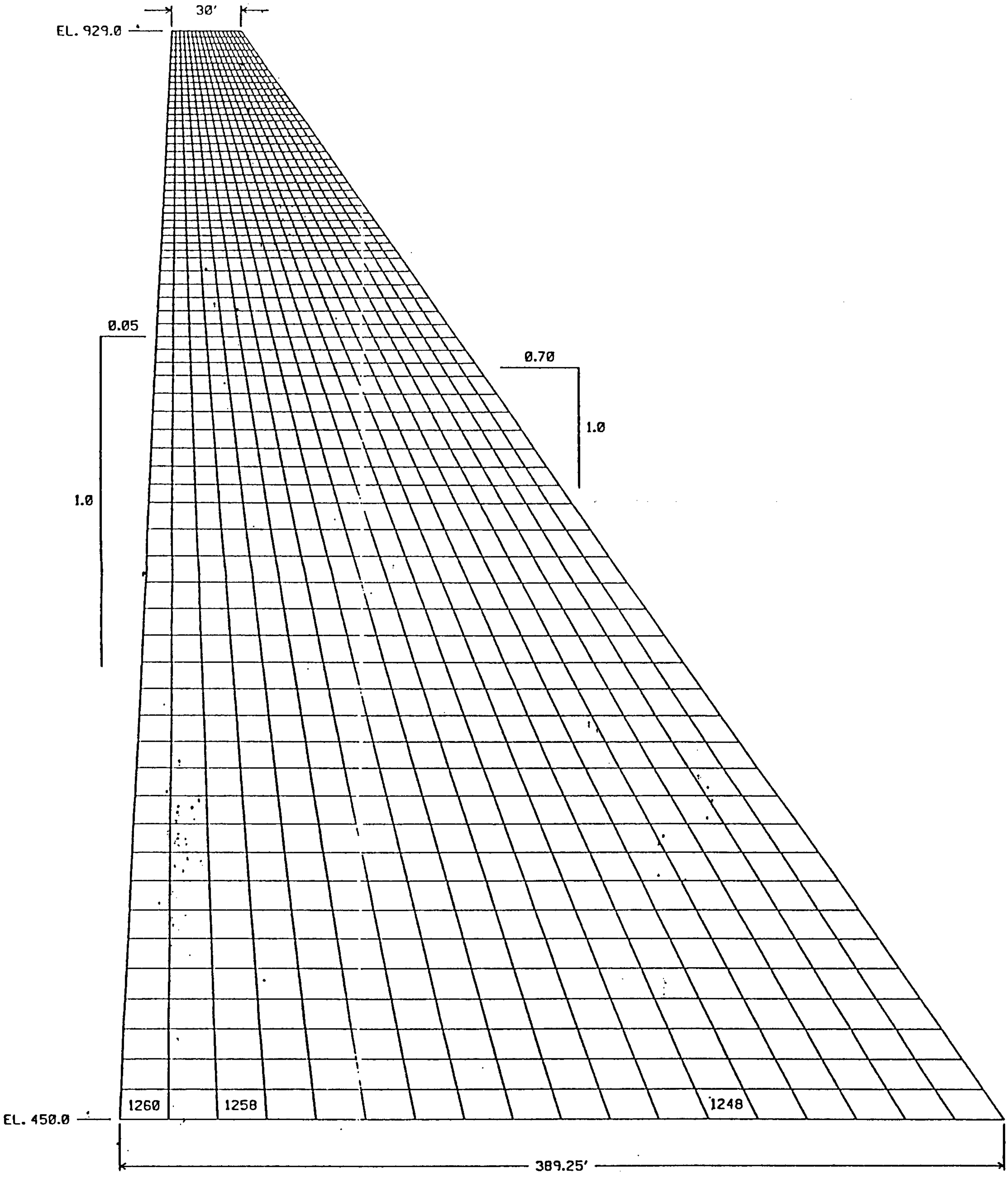
Foundation-rock Model

The foundation rock supporting the dam is idealized as a homogeneous, isotropic, viscoelastic half-plane [8,9,11,12]. This half-plane idealization permits accurate modeling of dam sites where uniform foundation materials extend to large depths under the dam. In order to define the dam-foundation system on a consistent basis, a plane stress model is employed for the foundation rock. This model, though not strictly appropriate for the foundation, is dictated by the expected behavior of the joints between the monoliths, as discussed above. For the purpose of including dam-foundation interaction effects, the foundation surface is assumed to be horizontal. The damsite in this study is idealized to conform to this assumption.



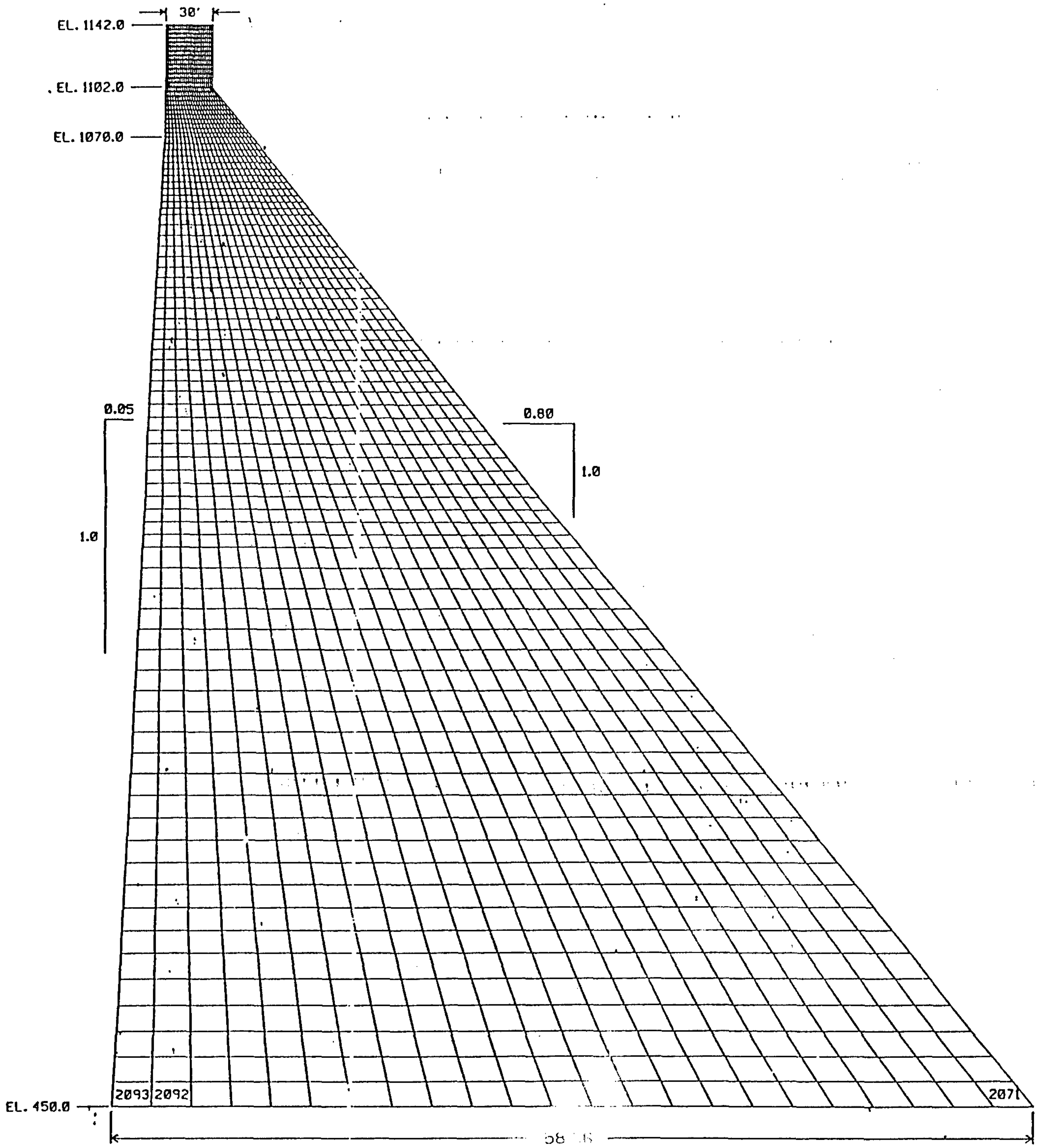
FINITE ELEMENT MESH OF THE PROPOSED AUBURN RCC GRAVITY DAM
200 YEAR FLOOD PROTECTION NON-EXPANDABLE DAM

FIGURE 2



FINITE ELEMENT MESH OF THE PROPOSED AUBURN RCC GRAVITY DAM
200 YEAR FLOOD PROTECTION EXPANDABLE DAM

FIGURE 3



FINITE ELEMENT MESH OF THE PROPOSED AUBURN RCC GRAVITY DAM
MULTIPURPOSE DAM

FIGURE 4

Hydrodynamic Effects

Hydrodynamic pressures in excess of the usual hydrostatic pressures are developed in the reservoir due to the earthquake ground motions and deformations of the upstream face of the dam. The structural deformations are in turn affected by the hydrodynamic pressures acting on the dam. The hydrodynamic effects need to be properly modeled to recognize the dynamic interaction between the dam and water. The analysis procedure implemented in EAGD-84 efficiently evaluates the dam-reservoir interaction during an earthquake. In this procedure, the water impounded in the reservoir is idealized as a continuum of constant depth and infinite length in the upstream direction [7,8,9]. The dynamic response of the dam with impounded water is affected to a significant degree by the hydrodynamic terms in the equations of motion for the dam. These terms are determined from the solution of the wave equation for appropriate accelerations at the boundaries of the fluid domain. The hydrodynamic terms can be interpreted as modifying the properties of the dam by introducing an added mass, an added force, and an added damping. The following properties for the impounded water are used: velocity of pressure waves in water = 4720 ft/sec, and unit weight = 62.4 lb/ft^3.

Material Properties

__Dam Concrete__. - The USBR conducted a concrete testing program for their authorized Auburn Dam in the 1970s. However, no laboratory and field tests on RCC were performed. Concrete properties for this analysis have been assumed based on the USBR's test data. In this study, the mass concrete in the dam is assumed to be homogeneous, isotropic, and linearly elastic with the following properties used for the dynamic analysis:

Young's modulus of elasticity Ec = 5000 ksi
Poisson's ratio Vc = 0.2
Unit weight Yc = 0.150 kip/ft^3

To evaluate the seismic performance of the proposed dam alternatives, concrete having an ultimate dynamic compressive strength of 6500 psi and a dynamic tensile strength of 750 psi was assumed. These strengths can reasonably be assumed for current RCC construction. Before final design is completed for any proposed dam project, mixture tests, field tests, and tests on existing RCC structures will be conducted to assure that these strengths can be achieved.

__Foundation Rock__. - No recommended value for the deformation modulus (or Young's modulus) of the foundation rock was provided in the geology reconnaissance report Appendix M, Chapter 5 prepared by the Corps of Engineers. Based on the USBR's report [6], a Young's modulus of

Elasticity Ef = 2500 ksi was selected for the finite element analysis. It was determined that small variation in the unit weight of the foundation rock has little effect on the dynamic response of the dam. Thus, a typical value for the unit weight, Yf = 0.165 kip/cu ft, was assumed in this investigation.

Seismic Design Event

The seismic design event used for the analysis of the alternatives is the one developed at the end of a tremendous amount of study by the USBR. The adopted event was the result of several eminent board's analysis and was adopted by both the Department of Interior and the California State Geologist. A chronology of the seismic studies and analyses accomplished is given in Chapter 5 of the Geotechnical Appendix. The adopted parameters are as follows:

1. A magnitude 6.5 Maximum Credible Earthquake with a response acceleration of 0.5g in the one second portion of the spectrum.

2. A fault slip in the foundation of up to 9 inches.

These parameters are considered conservative for this site. During detailed design, the previous studies will be reviewed with respect to any new methods of analysis or data to determine if the seismic design parameters should be modified.

Parameters for Earthquake Response Analysis

Constant Hysteretic Damping Factor. - Energy dissipation in the dam concrete is represented by constant hysteretic damping with a damping factor Nc. A viscous damping ratio E is taken to be the same for all the natural vibration modes of the dam supported on rigid foundation rock with no impounded water. This damping ratio E corresponds to a constant hysteretic damping factor given by Nc = 2E [8]. A constant hysteretic damping factor of Nc = 0.14, which corresponds to a 7 percent viscous damping ratio for all natural vibration modes of the dam on rigid foundation rock with empty reservoir, was selected. Nc = 0.14 is a reasonable value for the strong ground motion considered in this study and the relatively high stresses expected in the dam during the earthquake.

The constant hysteretic damping factor Nf for the foundation rock can be determined from experimental tests of appropriate rock samples subject to harmonically varying stress and strain. Since no experimental Nf value is available, Nf = 0.25 is assumed to represent the damping properties of the foundation rock. This damping factor is appropriate for the feasibility study.

Wave Reflection Coefficient. - The bottom of a reservoir typically consists of layers of alluvium, silt, and other sedimentary materials.

Hydrodynamic pressure waves impinging on such materials will partially reflect back into the water, and will partially be absorbed into the underlying layers of reservoir-bottom materials. In general, the dynamic response of the dam decreases with increasing wave absorption at the reservoir bottom.

The absorptiveness of the reservoir-bottom materials is characterized by the wave reflection coefficient alpha, which is defined as the ratio of the amplitude of the reflected hydrodynamic pressure wave to the amplitude of a normally propagating pressure wave incident on the reservoir bottom [9]. A rigid reservoir bottom (alpha = 1.0) indicates that pressures waves are reflected from the reservoir bottom without attenuation; whereas a completely absorptive reservoir bottom (alpha = 0) means that normally propagating pressure waves are fully absorbed into the reservoir bottom materials without reflection. The wave reflection coefficient alpha can be determined according to the following equation [9]:

$$\text{alpha} = (1-qC)/(1+qC)$$

in which C is the velocity of pressure waves in water, q is the damping coefficient of the reservoir-bottom materials and is given by

$$q = P/(Pr \quad Er/Pr)$$

where P is the mass density of water, Er is the Young's modulus of elasticity and Pr is the mass density of the materials at the reservoir bottom.

Taking Er = Ef = 2500 ksi for the foundation rock, the above equations lead to an alpha value of 0.65. Based on the recommendation from reference [13], alpha = 0.90 was used in the dynamic analysis of the alternative dams, as it provides conservative estimates of stresses in the dam.

Number of Vibration Modes. - To produce accurate earthquake response of the dam, all vibration modes that significantly contribute to the dynamic response should be included. Verification that enough vibration modes are included is by ascertaining that the stresses in the dam do not change if the number of modes is increased. In this investigation, 15 vibration modes were determined to be sufficient for accurate analysis of the non-expandable dam, expandable dam, and multipurpose dam. Computed natural frequencies for the first 10 modes of vibration of the dam-foundation rock system with empty reservoir are shown in Table N-3-9. EAGD-84 is unable to directly compute the resonant frequencies of the dam including dam-water interaction. These frequencies, which are known to be affected by dam-water interaction, have been investigated in depth in Dr. Fenves research study [9] and will not be re-investigated here.

TABLE N-3-9

NATURAL FREQUENCIES OF
DAM-FOUNDATION SYSTEM WITH NO IMPOUNDED WATER

Mode Number	Natural Frequencies in Hz Non-expandable Dam	Expandable Dam	Multipurpose Dam
Mode 1	2.09	2.06	1.64
Mode 2	4.84	4.52	3.49
Mode 3	5.60	5.78	4.25
Mode 4	8.80	9.59	6.60
Mode 5	13.01	13.90	9.36
Mode 6	14.90	15.30	10.64
Mode 7	17.88	18.57	12.28
Mode 8	20.21	19.00	13.00
Mode 9	21.97	21.92	14.89
Mode 10	23.65	24.29	15.90

Fourier Transform Parameters. - The earthquake response of the dam is obtained by Fourier synthesis of the complex-valued frequency response functions using the Fast Fourier Transform (FFT) algorithm. To ensure that EAGD-84 computes accurate dynamic response, the Fourier Transform parameters must be properly selected to satisfy the criteria summarized later in this report. The parameters used in the FFT computations are as follows:

Number of excitation frequencies N = 2048

Time interval for response history computation Dt = 0.01 sec.

Duration of response history T = N x Dt = 20.48 sec.

Maximum excitation frequency represented F = N / 2T = 50.0 Hz.

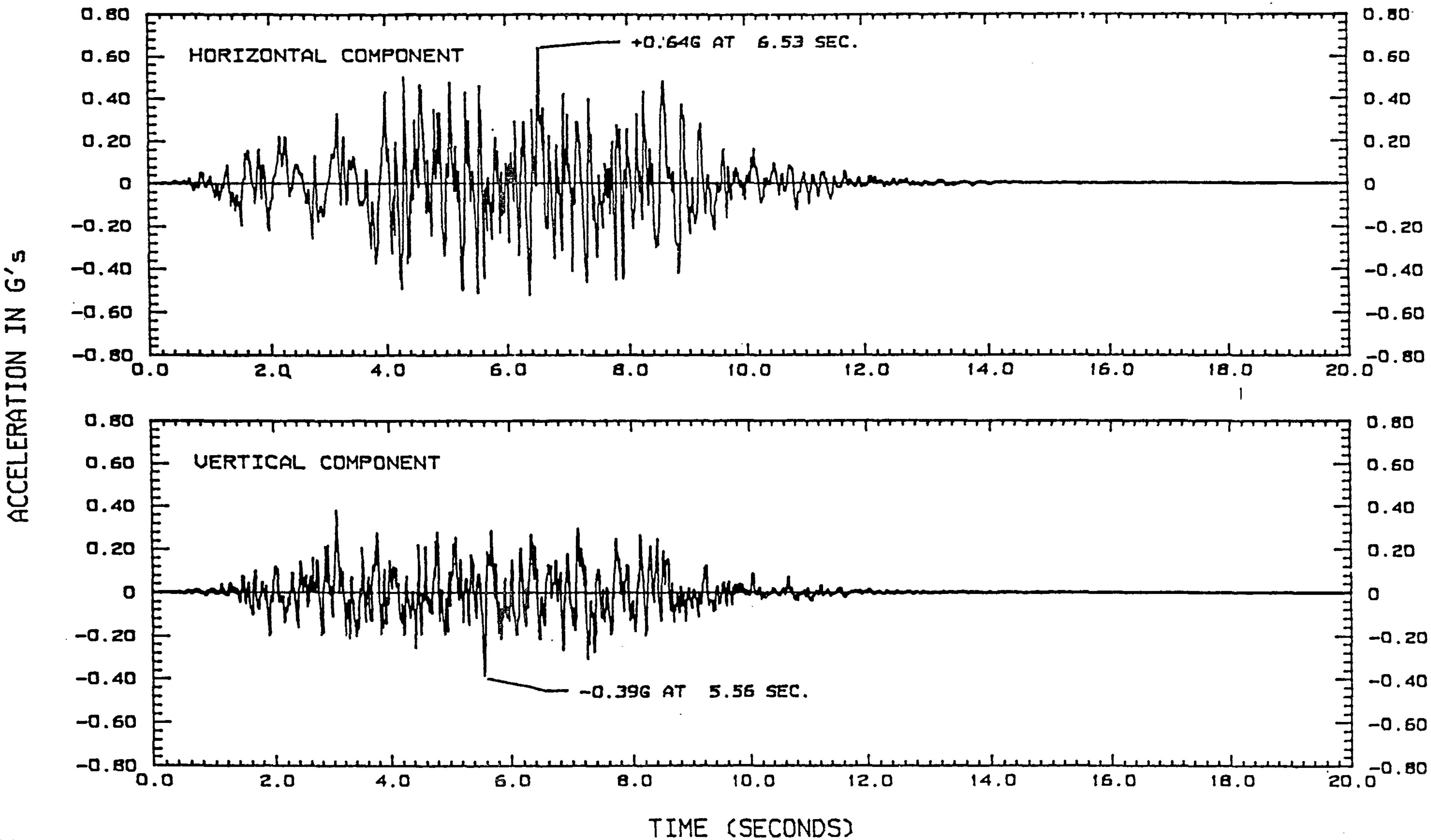
The following criteria [10] govern the selection of the above parameters:

- $Df \leq f1 / 50$ ....(1)
- $T > 1.5 / (Nc \times f1)$........(2)
- $F \leq 2.5Cf / (PI \times s)$.......(3)
- $F > Fh$ .... (4)

where Df is the frequency increment given by $Df = 1 / 2T$
f1 is the fundamental frequency of the dam-foundation rock system in Hertz
Nc is the constant hysteretic damping factor for the dam concrete
s is the distance between the equally spaced nodal points at the dam base
Fh is the frequency of the highest vibration mode included in the analysis
Cf is the shear wave velocity given by Gf/Pf, Gf is the elastic shear modulus and Pf is the mass density of the foundation rock

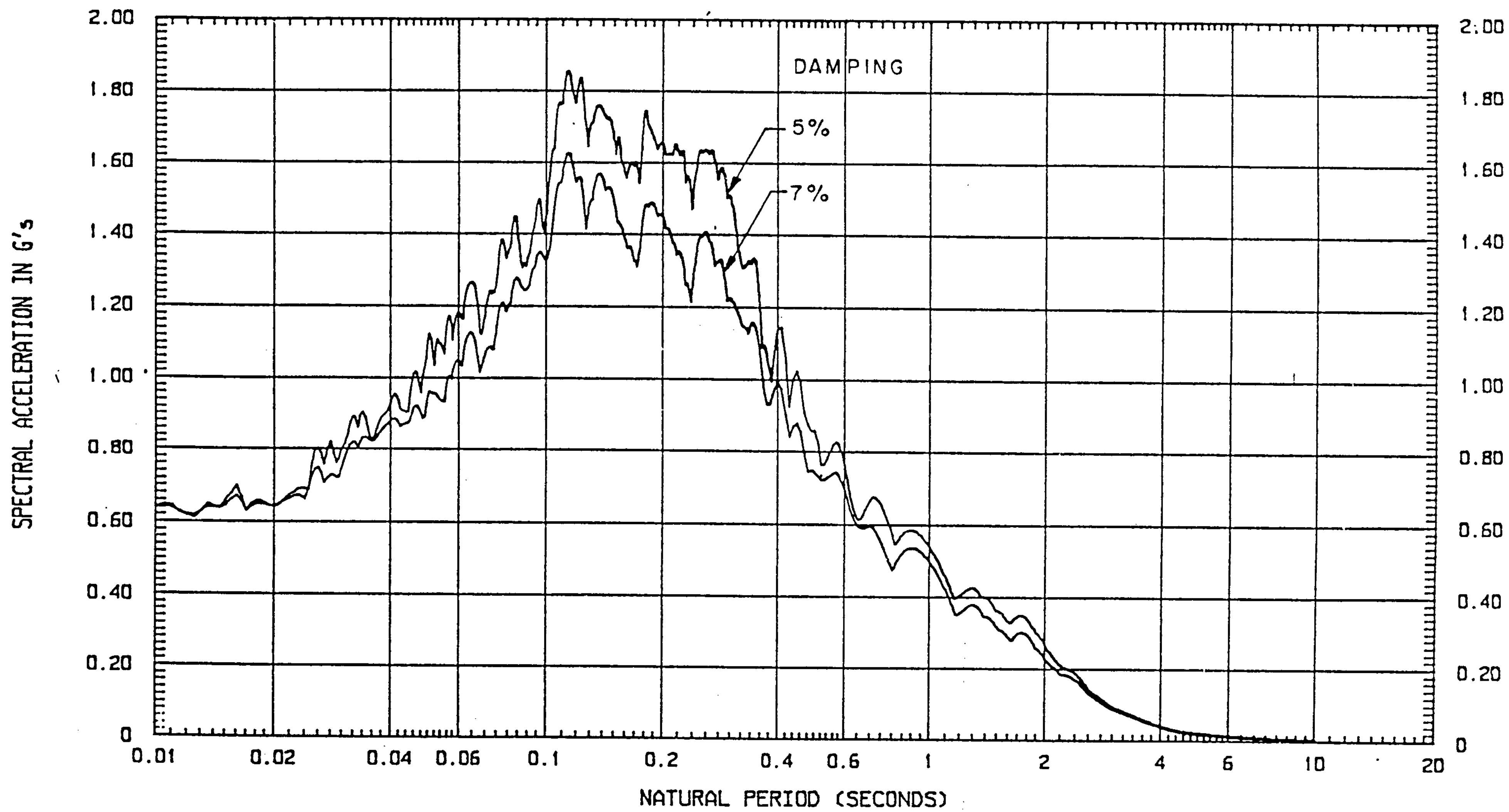
Earthquake Excitation. - The Bureau of Reclamation developed synthetic earthquake accelerograms used to model the Maximum Credible Earthquake (MCE) for the RM 20.1 damsite [6]. Basically, the accelerograms were generated to match the selected site response spectra. The time histories of ground acceleration in the horizontal (upstream-downstream) and vertical directions are shown in Figure 5. The response spectra for the horizontal and vertical components are displayed in Figures 6 and 7, respectively.

The earthquake excitation for the dam-water-foundation rock system is defined by the above two components of ground acceleration at the base of the dam: the horizontal component transverse to the dam axis, and the vertical component. This earthquake, having a peak acceleration of 0.64g in the horizontal direction and 0.39g in the vertical direction, provides an upper bound to feasible ground motions for this analysis.



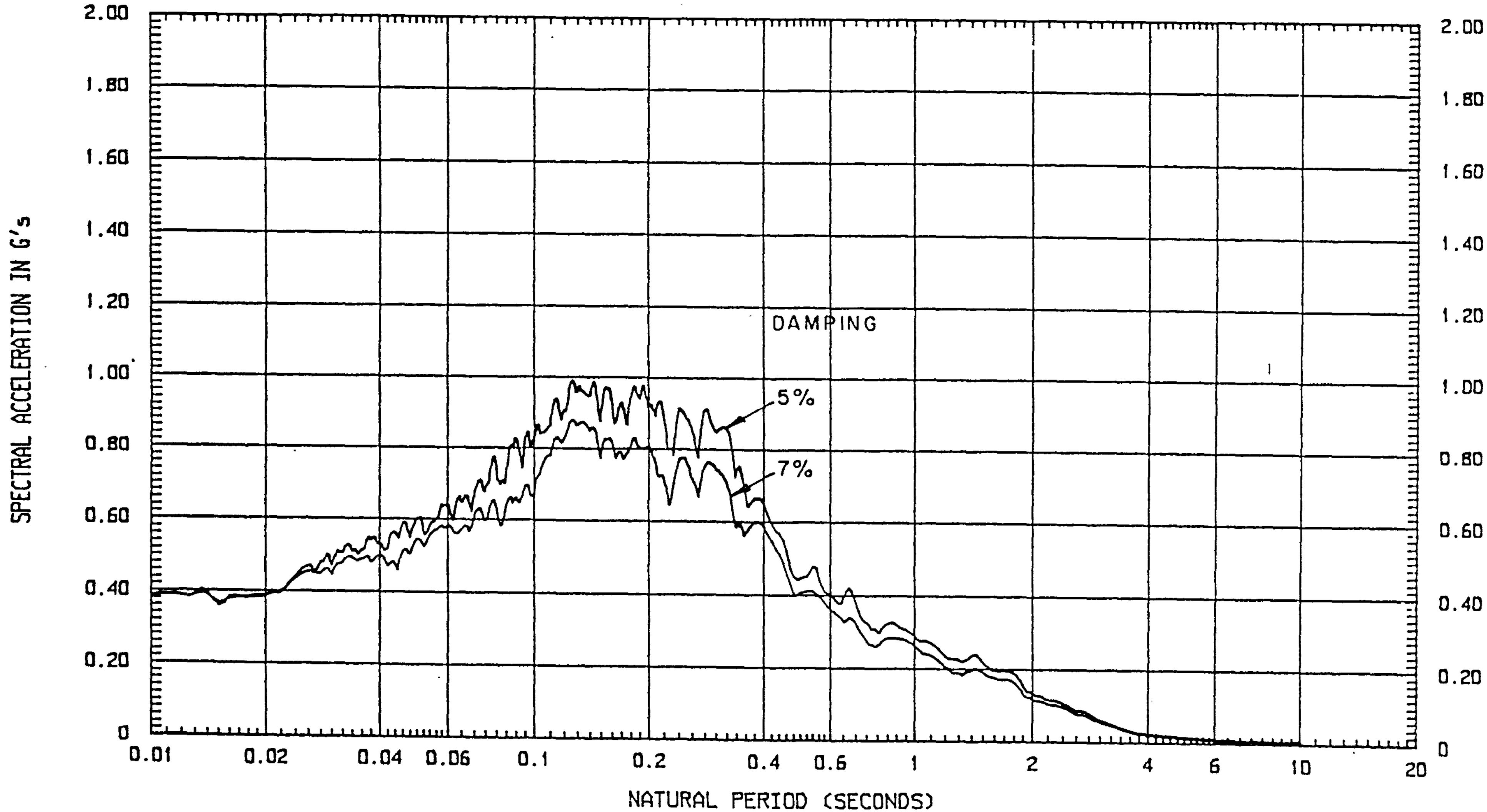
HORIZONTAL AND VERTICAL COMPONENTS OF THE MAXIMUM CREDIBLE EARTHQUAKE FOR SEISMIC ANALYSIS OF THE PROPOSED AUBURN DAM.

FIGURE 5



ACCELERATION RESPONSE SPECTRA FOR THE HORIZONTAL COMPONENT OF THE MAXIMUM CREDIBLE EARTHQUAKE. DAMPING RATIOS = 5 AND 7 PERCENT.

FIGURE 6



ACCELERATION RESPONSE SPECTRA FOR THE VERTICAL COMPONENT OF THE MAXIMUM CREDIBLE EARTHQUAKE. DAMPING RATIOS = 5 AND 7 PERCENT.

FIGURE 7

__Dynamic Response Results and Evaluation.__ - The earthquake response of the proposed dam was computed for the simultaneous excitation of the horizontal and vertical ground motions, under the following reservoir pool conditions:

a. spillway crest elevation 868.5 for the flood control only dam and the dam with future facilities.

b. normal pool elevation 1131.4 for the multipurpose dam, approximately 10 feet below the dam crest

Because of the immense volume of response results generated from the time-history earthquake analysis for different cases, only a small portion of the results is presented here to highlight the more important information. Stresses reported are total stresses computed at the centroid of the finite elements, including the initial static stresses due to dead weight of the dam and hydrostatic pressure.

Table N-3-10 summarizes the largest compressive stresses along with the corresponding stress locations and times of occurrence. Note that elements 18 and 1260 in Table N-3-10 correspond to the upstream heel area of the dam, and element 2071 corresponds to the downstream toe (see Figures 2, 3, and 4 for element numbers). These maximum compressive stresses are well within the assumed 6500 psi ultimate dynamic compressive strength, thus providing a large margin of safety against compressive failure.

TABLE N-3-10

SUMMARY OF MAXIMUM COMPRESSIVE STRESSES

	Finite Element	Time of Occurrence (Second)	Compressive Stress (psi)
Non-expandable dam	18	6.71	1218
Expandable dam	1260	6.71	1279
Multipurpose dam	2071	3.91	1605

The first two maximum principal (tensile) stresses along with the corresponding stress locations and times of occurrence are compiled in Table N-3-11.

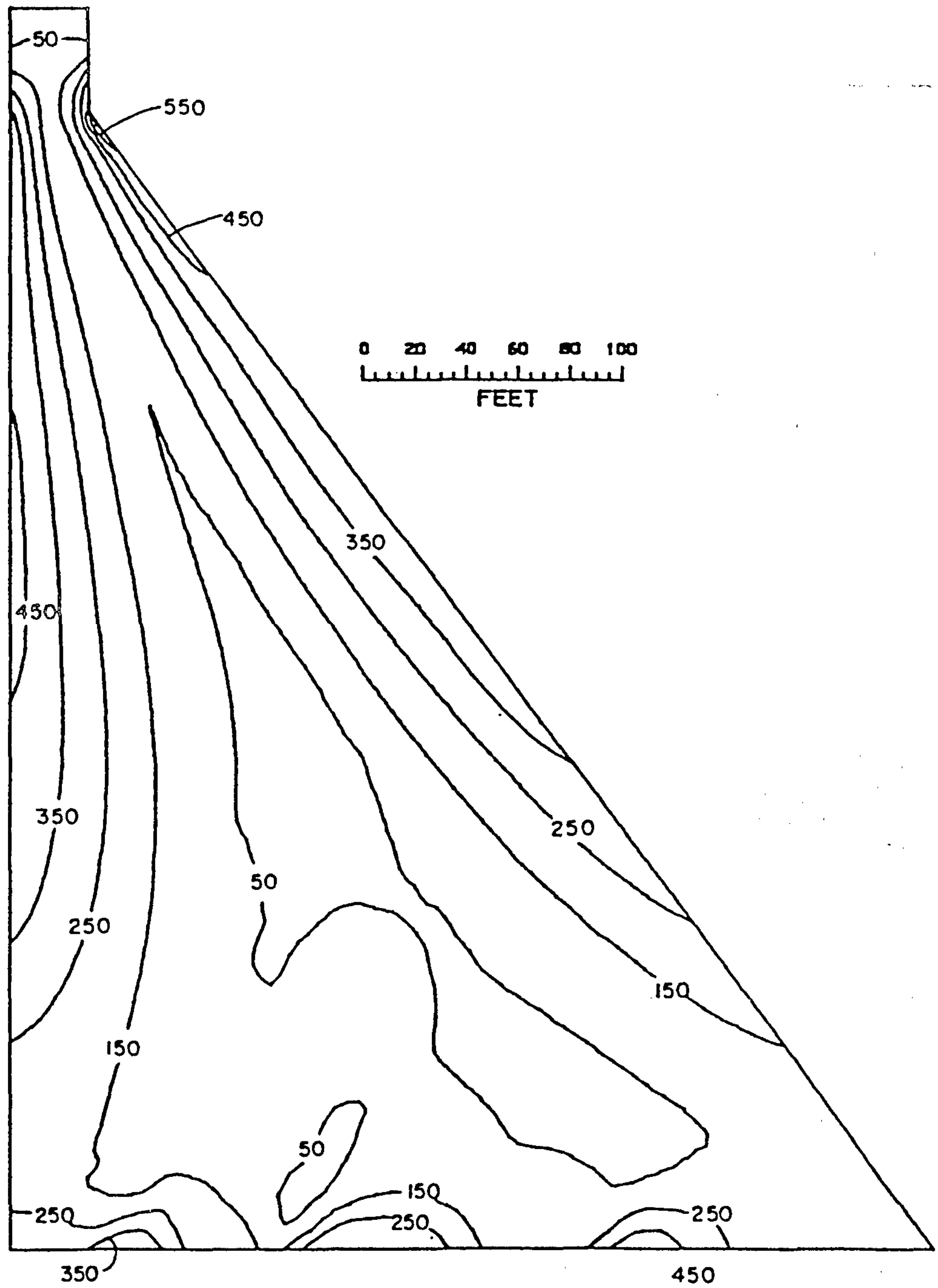
TABLE N-3-11

SUMMARY OF MAXIMUM TENSILE STRESSES

	Finite Element	Time of Occurrence (Second)	Tensile Stress (psi)
Non-expandable dam	937	5.21	561
	955	5.21	488
Expandable dam	1248	4.36	472
	1258	4.31	388
Multipurpose dam	2093	3.95	763
	2092	3.90	756

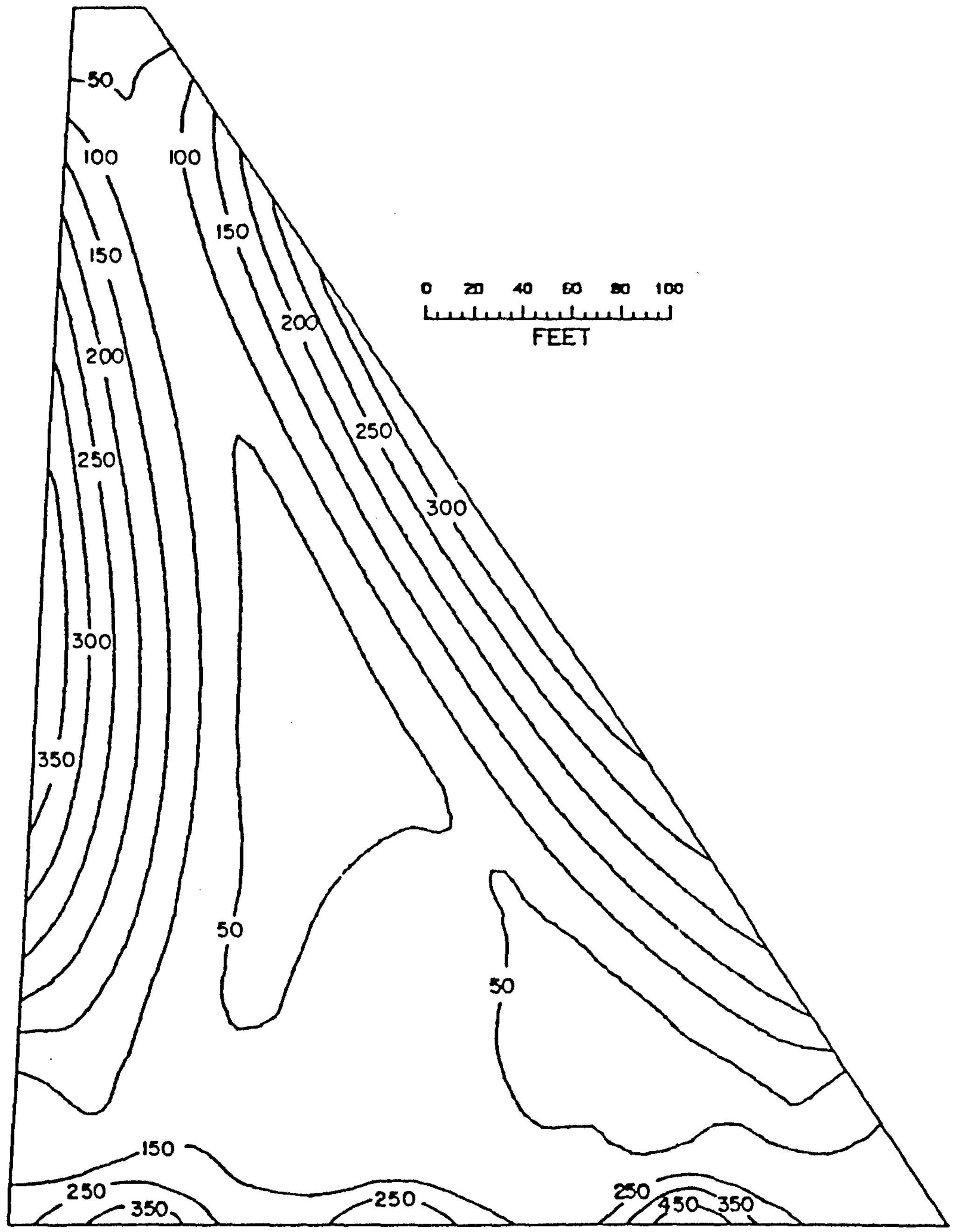
Stress contours are used to display the distribution of the maximum principal stresses in the non-expandable dam, expandable dam, and multipurpose dam. Contour plots of the envelope values of those stresses are shown in Figures 8, 9, and 10 ("envelope value" refers to the maximum value over time). Presented in Figures 11, 12, and 13 are time histories of the maximum principal stresses at the most-stressed locations, and in Figures 14, 15, and 16 are time histories of displacement response at the dam crest. For the non-expandable dam and expandable dam, the maximum tensile stresses are less than the assumed dynamic tensile strength of 750 psi. For the multipurpose dam, only two elements have tensile stresses slightly exceeding 750 psi for very brief instants of time (about 0.1 second), as can be seen from the stress time-history plot shown in Figure 13. These two elements correspond to the upstream heel of the dam. High tensile stresses in such localized area are acceptable even if some minor cracking can occur. Cracks, if developed, are not regarded as potentially damaging and will not impair the ability of the dam to contain the impounded water. On the basis of the overall stress levels, it can be expected that the dam response is predominantly linearly elastic, consistent with the assumption of the finite element analysis.

In conclusion, the alternative flood control only dams, dams with future facilities, and the authorized multipurpose dam are capable of withstanding the maximum credible earthquake and post-earthquake loads (hydrostatic pressure and dead weight of dam) in such a way that no failure triggering a sudden, catastrophic release of water will occur, and that life and property downstream will not be endangered.



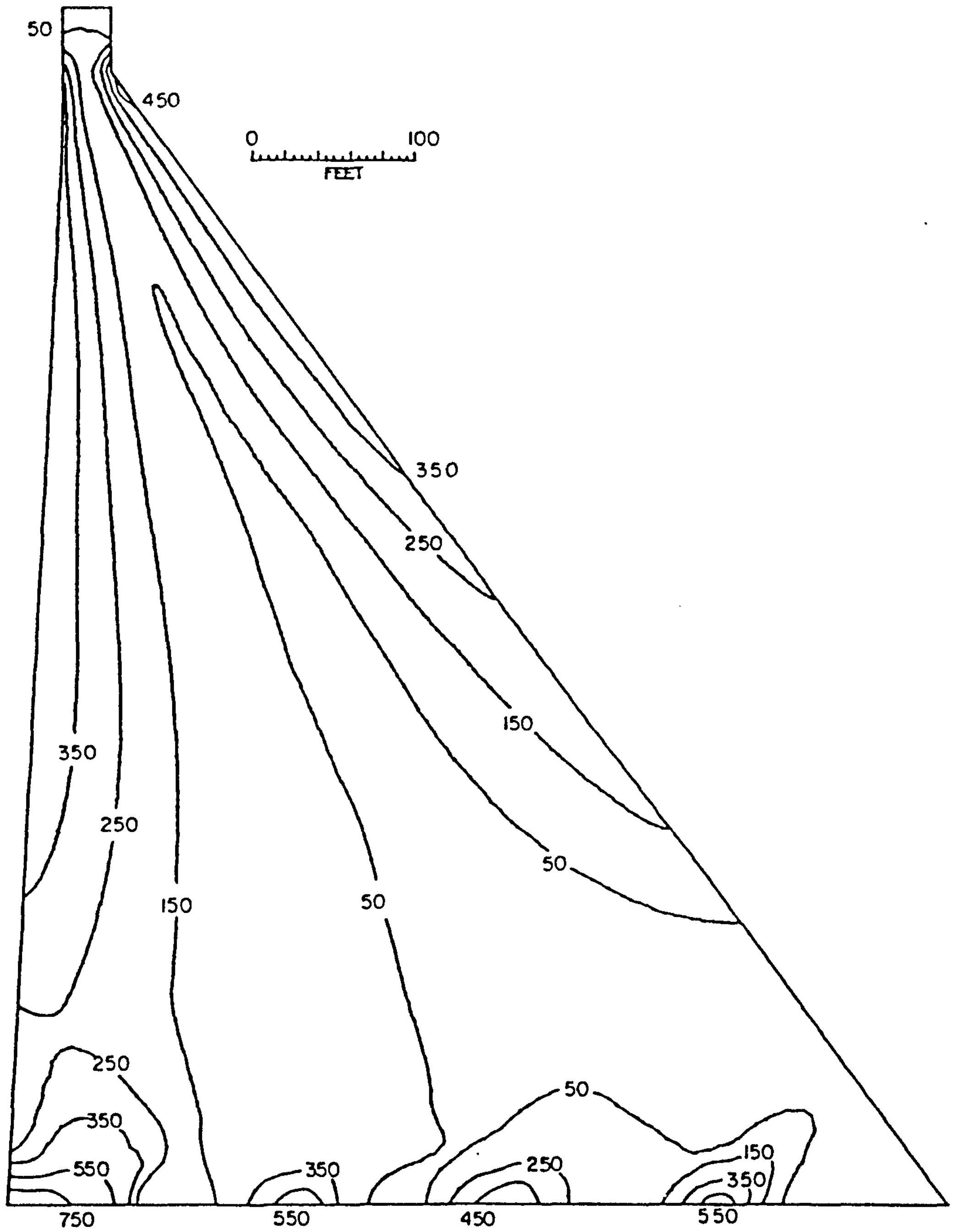
ENVELOPE VALUES OF MAXIMUM PRINCIPAL STRESSES (IN PSI) IN THE PROPOSED NON-EXPANDABLE AUBURN DAM WITH POOL ELEVATION = 868.5 FEET. INITIAL STATIC STRESSES ARE INCLUDED.

FIGURE 8



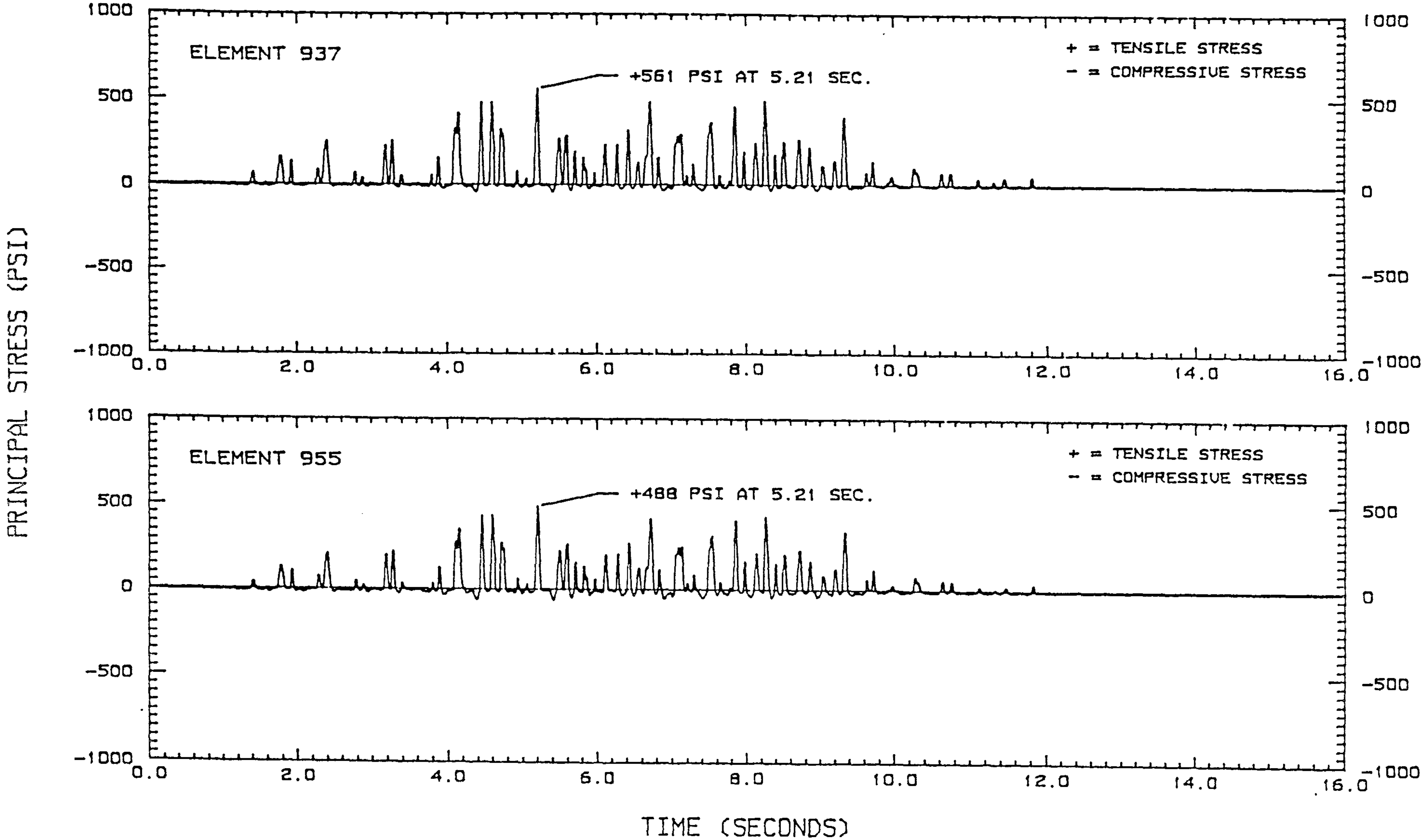
ENVELOPE VALUES OF MAXIMUM PRINCIPAL STRESSES (IN PSI) IN THE PROPOSED EXPANDABLE AUBURN DAM WITH POOL ELEVATION = 868.5 FEET. INITIAL STATIC STRESSES ARE INCLUDED.

FIGURE 9



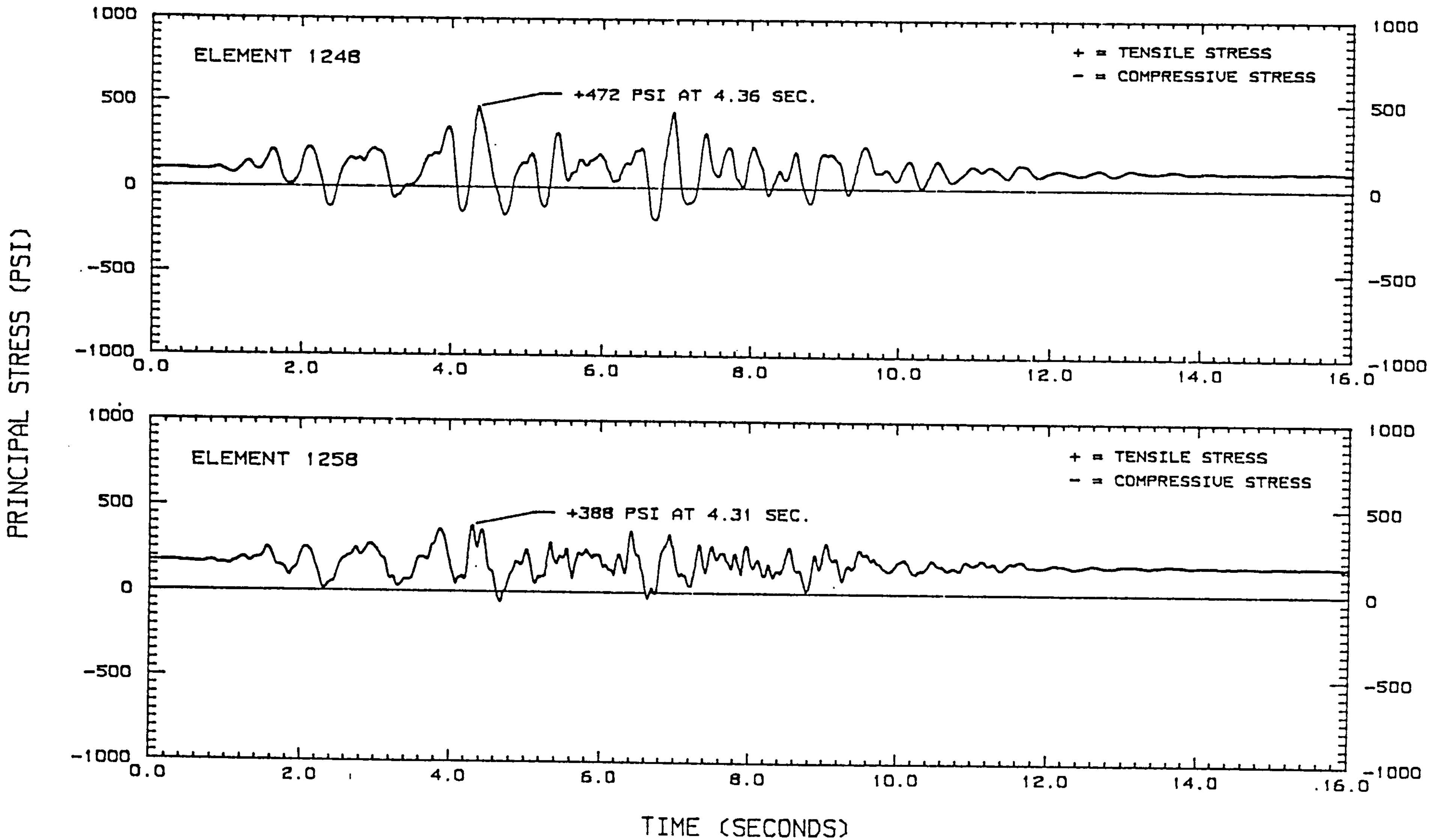
ENVELOPE VALUES OF MAXIMUM PRINCIPAL STRESSES (IN PSI) IN THE PROPOSED MULTIPURPOSE AUBURN DAM WITH POOL ELEVATION = 1131.4 FEET. INITIAL STATIC STRESSES ARE INCLUDED.

FIGURE 10



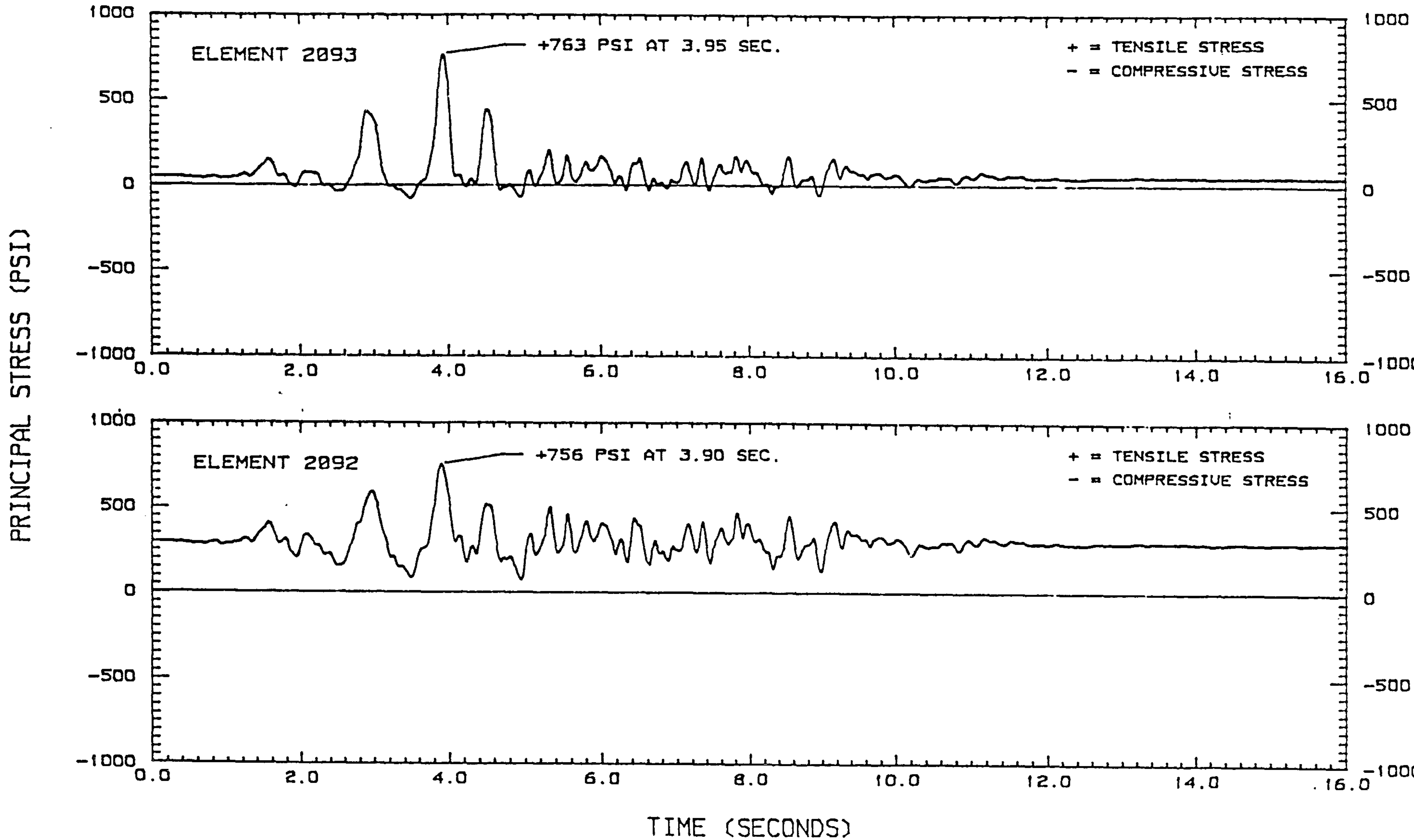
STRESS RESPONSE AT SELECTED LOCATIONS OF THE PROPOSED NON-EXPANDABLE AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.

FIGURE 11



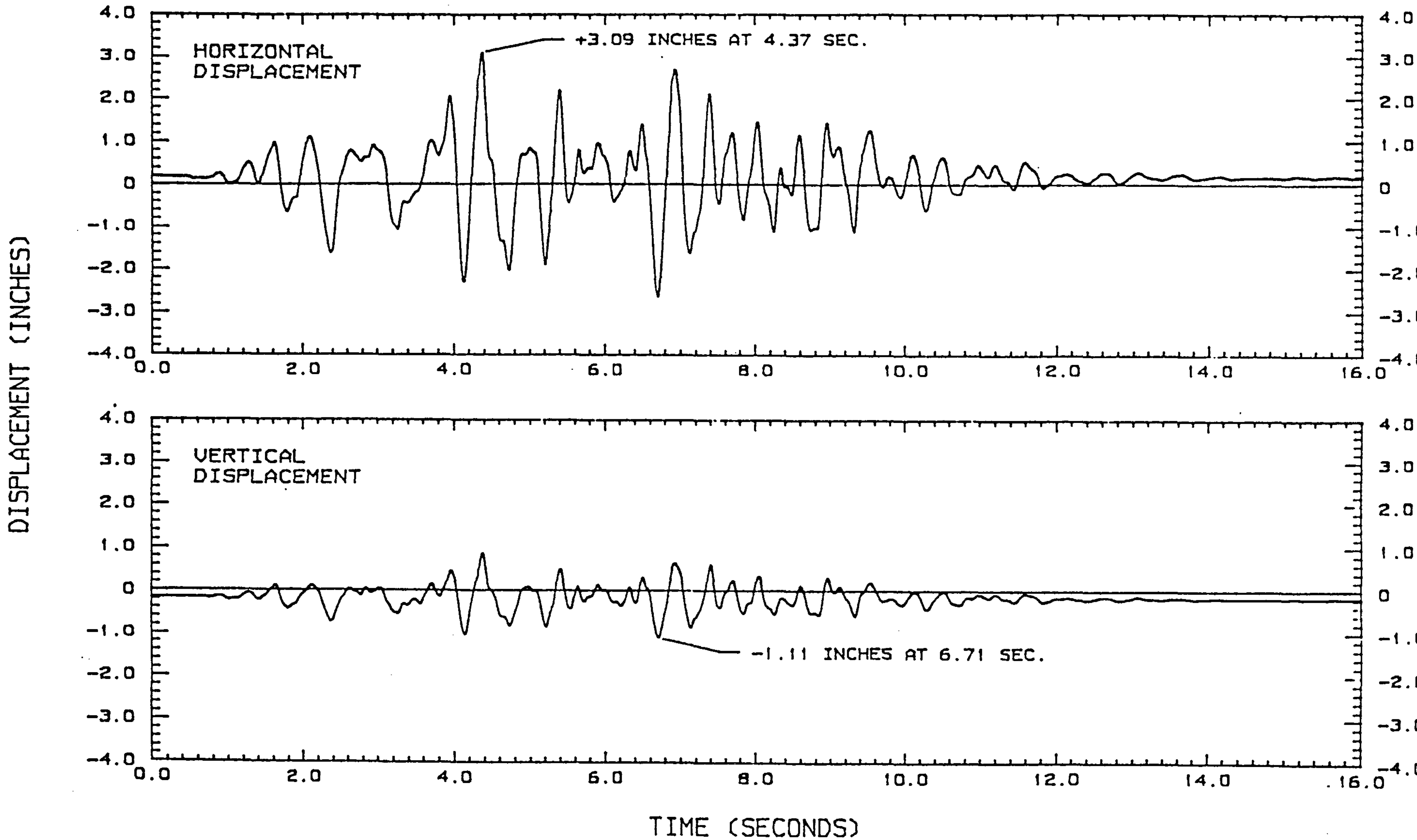
STRESS RESPONSE AT SELECTED LOCATIONS OF THE PROPOSED EXPANDABLE AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.

FIGURE 12



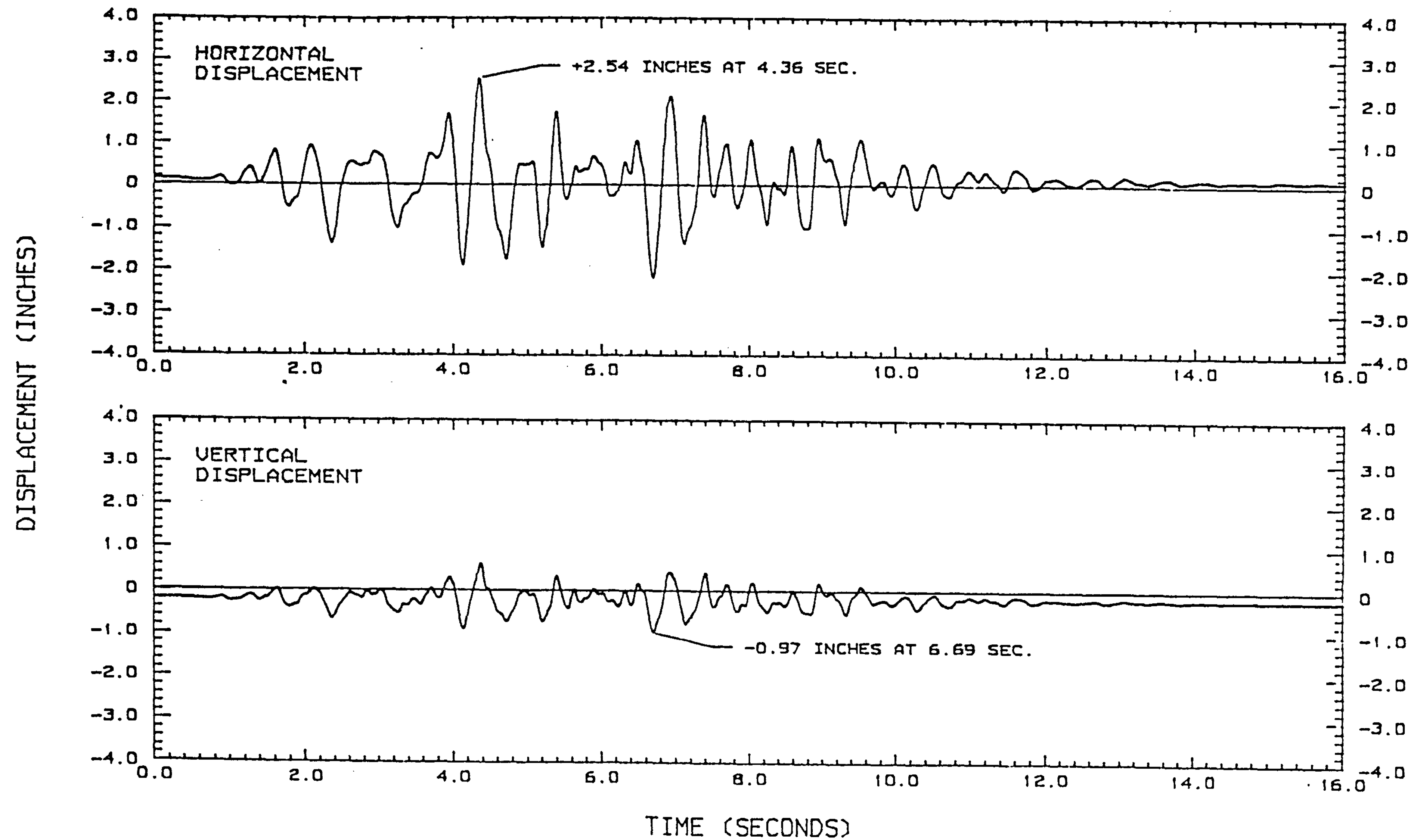
STRESS RESPONSE AT SELECTED LOCATIONS OF THE PROPOSED MULTIPURPOSE AUBURN DAM WITH POOL ELEVATION = 1131.4 FEET.

FIGURE 13



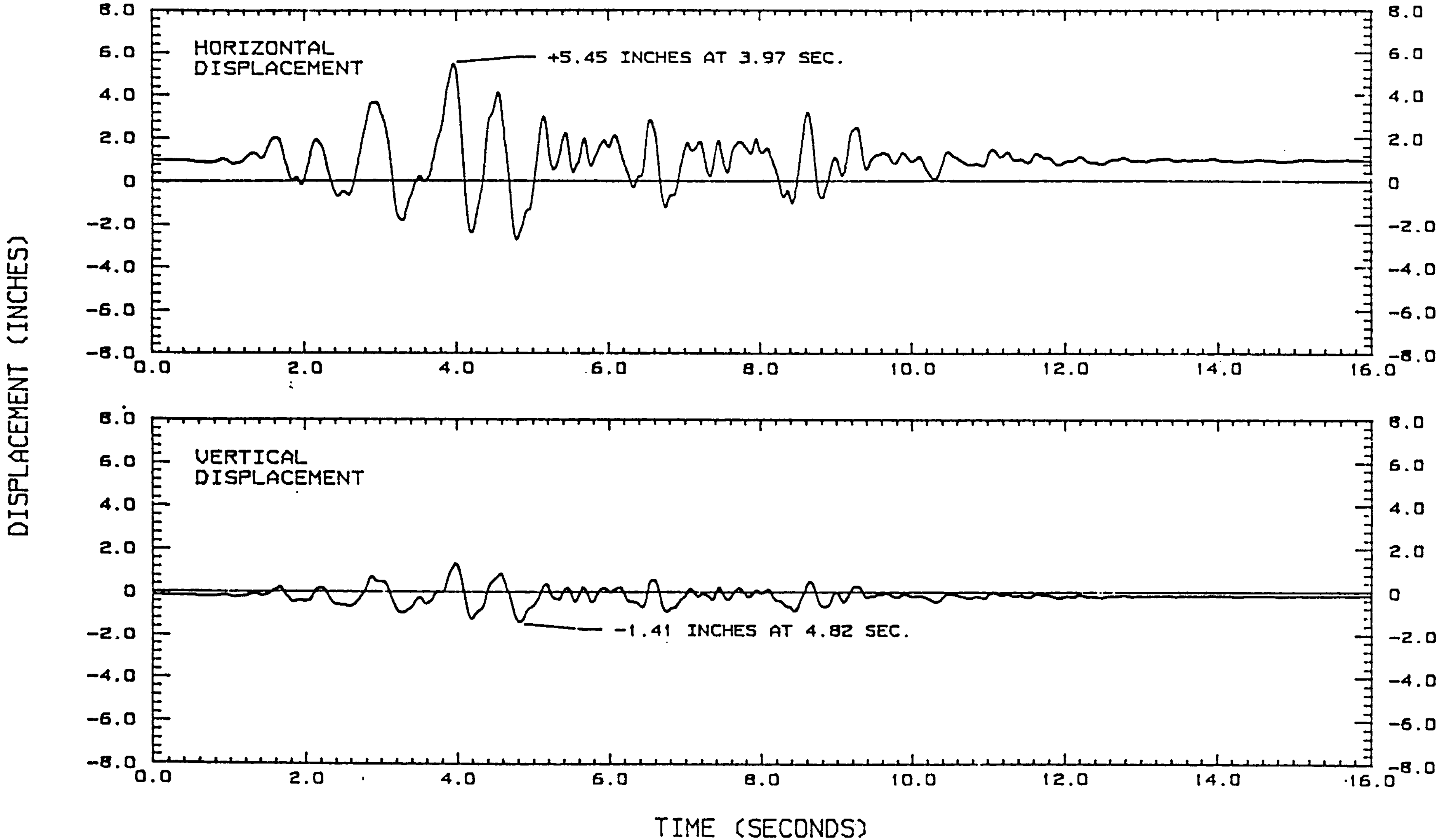
DISPLACEMENT RESPONSE AT DAM CREST OF THE PROPOSED NON-EXPANDABLE AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.

FIGURE 14



DISPLACEMENT RESPONSE AT DAM CREST OF THE PROPOSED EXPANDABLE AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.

FIGURE 15



DISPLACEMENT RESPONSE AT DAM CREST OF THE PROPOSED MULTIPURPOSE
AUBURN DAM WITH POOL ELEVATION = 1131.4 FEET.

FIGURE 16

ALTERNATIVE COSTS

The cost estimates developed for the individual dam alternatives analyzed are given in Tables N-3-12 thru N-3-18. Table N-3-19 shows a comparison of the seven alternatives.

TABLE N-3-12

COST ESTIMATE
100-YEAR FLOOD CONTROL ONLY
ROLLER COMPACTED CONCRETE DAM
RIVER MILE 20.1
OCTOBER 91 PRICE LEVEL

ITEM	DESCRIPTION	COST
01	LANDS	
	RESERVOIR LANDS	23,400,000
	HIGHWAY 49 ROW	15,400,000
	TOTAL	38,800,000
02	RELOCATIONS	
	HIGHWAY 49	77,200,000
	PONDEROSA WAY	11,100,000
	UTILITIES	2,200,000
	CONTINGENCIES	17,500,000
	TOTAL	108,000,000
04	DAM	
	MOBILIZATION	1,700,000
	EXCAVATION & FOUNDATION WORK	27,400,000
	EMBANKMENT	70,600,000
	SPILLWAY	6,400,000
	SLUICES	7,300,000
	DIVERSION TUNNEL	13,500,000
	GALLERIES	2,400,000
	CONTINGENCIES	25,100,000
	TOTAL	154,400,000
08	ROADS	
	ACCESS ROADS	2,000,000
30	ENGINEERING AND DESIGN	31,600,000
31	SUPERVISION & INSPECTION	21,100,000
	PROJECT TOTAL	355,900,000

TABLE N-3-13

COST ESTIMATE
200-YEAR FLOOD CONTROL ONLY
ROLLER COMPACTED CONCRETE DAM
RIVER MILE 20.1
OCTOBER 91 PRICE LEVEL

ITEM	DESCRIPTION	COST
01	LANDS	
	RESERVOIR LANDS	28,000,000
	HIGHWAY 49 ROW	15,400,000
	TOTAL	43,400,000
02	RELOCATIONS	
	HIGHWAY 49	77,200,000
	PONDEROSA WAY	11,100,000
	UTILITIES	2,200,000
	CONTINGENCIES	17,500,000
	TOTAL	108,000,000
04	DAM	
	MOBILIZATION	1,700,000
	EXCAVATION & FOUNDATION WORK	37,300,000
	EMBANKMENT	111,500,000
	SPILLWAY	8,300,000
	SLUICES	3,100,000
	DIVERSION TUNNEL	13,500,000
	GALLERIES	2,500,000
	CONTINGENCIES	34,400,000
	TOTAL	212,300,000
08	ROADS	
	ACCESS ROADS	2,000,000
30	ENGINEERING AND DESIGN	38,600,000
31	SUPERVISION & INSPECTION	25,700,000
	PROJECT TOTAL	430,000,000

TABLE N-3-14

COST ESTIMATE
400-YEAR FLOOD CONTROL ONLY
ROLLER COMPACTED CONCRETE DAM
RIVER MILE 20.1
OCTOBER 91 PRICE LEVEL

ITEM	DESCRIPTION	COST
01	LANDS	
	RESERVOIR LANDS	33,800,000
	HIGHWAY 49 ROW	15,400,000
	TOTAL	49,200,000
02	RELOCATIONS	
	HIGHWAY 49	77,200,000
	PONDEROSA WAY	11,100,000
	UTILITIES	2,200,000
	CONTINGENCIES	17,500,000
	TOTAL	108,000,000
04	DAM	
	MOBILIZATION	1,700,000
	EXCAVATION & FOUNDATION WORK	44,900,000
	EMBANKMENT	155,100,000
	SPILLWAY	8,700,000
	SLUICES	2,600,000
	DIVERSION TUNNEL	13,500,000
	GALLERIES	3,000,000
	CONTINGENCIES	44,500,000
	TOTAL	274,000,000
08	ROADS	
	ACCESS ROADS	2,100,000
30	ENGINEERING AND DESIGN	45,900,000
31	SUPERVISION & INSPECTION	30,600,000
	PROJECT TOTAL	509,800,000

TABLE N-3-15

COST ESTIMATE
100-YEAR FLOOD CONTROL WITH FUTURE FACILITIES
ROLLER COMPACTED CONCRETE DAM
RIVER MILE 20.1
OCTOBER 91 PRICE LEVEL

ITEM	DESCRIPTION	COST
01	LANDS	
	RESERVOIR LANDS	85,100,000
	HIGHWAY 49 ROW	15,400,000
	TOTAL	100,500,000
02	RELOCATIONS	
	HIGHWAY 49	77,200,000
	PONDEROSA WAY	11,100,000
	UTILITIES	2,200,000
	CONTINGENCIES	17,500,000
	TOTAL	108,000,000
04	DAM	
	MOBILIZATION	1,700,000
	EXCAVATION & FOUNDATION WORK	27,700,000
	EMBANKMENT	72,800,000
	SPILLWAY	6,100,000
	SLUICES	1,500,000
	PENSTOCKS	11,400,000
	DIVERSION TUNNEL	13,500,000
	GALLERIES	6,600,000
	CONTINGENCIES	27,400,000
	TOTAL	168,700,000
08	ROADS	
	ACCESS ROADS	2,000,000
30	ENGINEERING AND DESIGN	33,400,000
31	SUPERVISION & INSPECTION	22,300,000
	PROJECT TOTAL	434,900,000

TABLE N-3-16

COST ESTIMATE
200-YEAR FLOOD CONTROL WITH FUTURE FACILITIES
ROLLER COMPACTED CONCRETE DAM
RIVER MILE 20.1
OCTOBER 91 PRICE LEVEL

ITEM	DESCRIPTION	COST
01	LANDS	
	RESERVOIR LANDS	85,100,000
	HIGHWAY 49 ROW	15,400,000
	TOTAL	100,500,000
02	RELOCATIONS	
	HIGHWAY 49	77,200,000
	PONDEROSA WAY	11,100,000
	UTILITIES	2,200,000
	CONTINGENCIES	17,500,000
	TOTAL	108,000,000
04	DAM	
	MOBILIZATION	1,700,000
	EXCAVATION & FOUNDATION WORK	36,200,000
	EMBANKMENT	113,700,000
	SPILLWAY	8,100,000
	SLUICES	1,800,000
	PENSTOCKS	12,900,000
	DIVERSION TUNNEL	13,500,000
	GALLERIES	7,300,000
	CONTINGENCIES	37,800,000
	TOTAL	233,000,000
08	ROADS	
	ACCESS ROADS	2,000,000
30	ENGINEERING AND DESIGN	41,100,000
31	SUPERVISION & INSPECTION	27,300,000
	PROJECT TOTAL	511,900,000

TABLE N-3-17

COST ESTIMATE
400-YEAR FLOOD CONTROL WITH FUTURE FACILITIES
ROLLER COMPACTED CONCRETE DAM
RIVER MILE 20.1
OCTOBER 91 PRICE LEVEL

ITEM	DESCRIPTION	COST
01	LANDS	
	RESERVOIR LANDS	85,100,000
	HIGHWAY 49 ROW	15,400,000
	TOTAL	100,500,000
02	RELOCATIONS	
	HIGHWAY 49	77,200,000
	PONDEROSA WAY	11,100,000
	UTILITIES	2,200,000
	CONTINGENCIES	17,500,000
	TOTAL	108,000,000
04	DAM	
	MOBILIZATION	1,700,000
	EXCAVATION & FOUNDATION WORK	44,000,000
	EMBANKMENT	156,400,000
	SPILLWAY	8,600,000
	SLUICES	2,000,000
	PENSTOCKS	14,200,000
	DIVERSION TUNNEL	13,500,000
	GALLERIES	7,900,000
	CONTINGENCIES	48,000,000
	TOTAL	296,300,000
08	ROADS	
	ACCESS ROADS	2,100,000
30	ENGINEERING AND DESIGN	48,600,000
31	SUPERVISION & INSPECTION	32,400,000
	PROJECT TOTAL	587,900,000

TABLE N-3-18

COST ESTIMATE
200-YEAR MINIMUM POOL ALTERNATIVE
ROLLER COMPACTED CONCRETE DAM
RIVER MILE 20.1
OCTOBER 91 PRICE LEVEL

ITEM	DESCRIPTION	COST
01	LANDS	
	RESERVOIR LANDS	37,300,000
	HIGHWAY 49 ROW	15,400,000
	TOTAL	52,700,000
02	RELOCATIONS	
	HIGHWAY 49	77,200,000
	PONDEROSA Y 0	11,100,000
	UTILITIES	2,200,000
	CONTINGENCIES	17,500,000
	TOTAL	108,000,000
03	RESERVOIRS	4,000,000
	CONTINGENCIES	1,100,000
	TOTAL	5,100,000
04	DAM	
	MOBILIZATION	1,700,000
	EXCAVATION & FOUNDATION WORK	40,200,000
	EMBANKMENT	136,400,000
	SPILLWAY	8,600,000
	SLUICES	13,000,000
	PENSTOCKS	10,000,000
	DIVERSION TUNNEL	1,100,000
	GALLERIES	8,700,000
	CONTINGENCIES	42,600,000
	TOTAL	262,300,000
08	ROADS	
	ACCESS ROADS	2,000,000
14	MINIMUM RECREATION FACILITIES	400,000
	CONTINGENCIES	100,000
	TOTAL	500,000
30	ENGINEERING AND DESIGN	45,200,000
31	SUPERVISION & INSPECTION	30,200,000
	PROJECT TOTAL	506,000,000

TABLE N-3-19

DAM ALTERNATIVES
COST COMPARISONS
($1000)
OCTOBER 1991 PRICE LEVEL

	100-YR FC ONLY	200-YR FC ONLY	400-YR FC ONLY	100-YR FUTURE FACILITIES	200-YR FUTURE FACILITIES	400-YR FUTURE FACILITIES	MIN POOL
01 LANDS	38,800	43,400	49,200	100,500	100,500	100,500	52,700
02 RELOCATIONS	108,000	108,000	108,000	108,000	108,000	108,000	108,000
03 RESERVOIR	0	0	0	0	0	0	5,100
04 DAM	154,400	212,300	274,000	168,700	233,000	296,300	262,300
08 ROADS	2,000	2,000	2,100	2,000	2,000	2,100	2,000
14 RECREATION FACILITIES	0	0	0	0	0	0	500
30 E&D	31,600	38,600	45,900	33,400	41,100	48,600	45,200
31 S&A	21,100	25,700	30,600	22,300	27,300	32,400	30,200
PROJECT TOTAL	355,900	430,000	509,800	434,900	511,900	587,900	506,000

SELECTED PLAN DESCRIPTION

The selected plan for the dam is the 200-year flood control only dam. Several differences exist between the alternatives analyzed and the selected plan and will be discussed later in this section. More detailed analysis of foundation conditions and requirements were conducted for the selected plan. This further detailed analysis is required because of the concern of the public with a dam in this seismic area, to better define quantities and designs for the selected plan, and to more accurately establish the cost estimate for this project. The more detailed analysis has resulted in some aspects of the selected plan being different than what was determined for the alternatives analyzed. Table N-3-20 gives the pertinent data for the dam portion of the selected plan. Plate 7 shows the site plan and plates 8 and 9 show the profile and dam sections for the selected plan. The following information provides further details on the selected plan design.

Alignment

After additional consideration of the seismic conditions at the site it has been determined that, for a concrete dam, a slightly curved alignment would be more prudent than a straight one. Coordination with the California Department of Water Resources, Division of Safety of Dams, found that they agree with this conclusion. A curved alignment would further help to insure that there would be no monolith displacement downstream should an earthquake occur during flood conditions. The curved alignment is very gradual and does not significantly increase the dam crest length. Based on these considerations, the selected plan for the dam does include a slight upstream curve in the alignment.

Alignment studies for the selected plan included three curved alignments at the RM 20.1 site, designated C-1 through C-3 from upstream to downstream. To assess the feasibility of each of the alignments numerous geologic factors were taken into account. Each of the alignments were evaluated using the assumption that the ultimate end-product could be a multipurpose dam with the top of dam around elevation 1140 feet. Major factors included in the determination were the extent of geologic explorations previously conducted, the position of the alignment relative to Fault F-1 (the major known existing fault in the immediate damsite area), the large landslide (Slide 16) in the right canyon wall, the desire to site the spillway in the center of the canyon, and the excavation and foundation treatment already completed for the Bureau of Reclamation's authorized thin-arch dam. Because of the close proximity of the three alignments to one another, and the stage of study, the C-2 alignment was disregarded for this phase of the study.

TABLE N-3-20

PERTINENT DATA
SELECTED PLAN
DAM AT RIVER MILE 20.1

1. General Data

Stream	N Fork American River
River Mile	20.1
Purpose	Flood Control
Drainage Area at Dam Site	973 Sq. Miles

2. Reservoir Data

Elevation:	
Gross Pool (Spillway Crest)	868.5 NGVD
Spillway Design Flood Pool S.D.F.P.	923.7 NGVD
Area:	
Gross Pool (Spillway Crest)	4,000 Acres
Spillway Design Flood Pool	5,100 Acres
Storage Capacity:	
Gross Pool (Spillway Crest)	545,000 Acre-Feet
Spillway Design Flood Pool	795,000 Acre-Feet

3. Probable Maximum Flood

Total Volume Runoff (5-Day Volume)	1,854,000 Acre-Feet
Peak Inflow	1,070,000 cfs
Peak Outflow	860,000 cfs

4. Dam

Type	Roller Compacted Concrete Gravity
Crest Elevation	923.7 NGVD
Freeboard Above S.D.F.P.	3.0 Feet (Parapet Wall)
Maximum Height	425 Feet 1/
Slopes	
Upstream	0.05H:1V
Downstream	0.70H:1V
Crest Length	2600 Feet
Crest Width	25 Feet
Total Volume of Concrete	5,200,000 CY

1/ Measured from streambed elevation of 498.7 feet.

TABLE N-3-20 (Cont'd)

5. Spillway

Type	Ungated, Ogee Weir
Crest Elevation (Gross Pool El.)	868.5
Crest Length	600 Feet
Surcharge Head:	
At Gross Pool Elevation	0
At S.D.F.P.	55.2 Feet
Design Discharge at S.D.F.P.	860,000 cfs

6. Outlet Works

Outlet Type	12 - rectangular sluices 6 at each side of spillway, provided with emergency gates
Sluice Size	5.0 Feet Wide by 9.5 Feet High
Intake Elevation	2 at 491 Feet NGVD 10 at 500 Feet NGVD
Length of Sluice	400 Feet
Design Discharge	7,250 cfs each sluice
Design Pool Elevation	868.5 Feet

Preliminary studies were concentrated on upstream alignment C-1. This alignment was designed so that its right abutment utilized the massive concrete dental work on the "J" block of the USBR's thin-arch foundation. By utilizing the work already completed on the "J" block, it was thought that significant savings could be accomplished. Further analysis indicated that due to adverse slopes in the area, a significant amount of rock would still need to be removed to provide an adequate foundation for the C-1 alignment. In addition, it was felt that the upper left abutment, while not crossing the surface trace of Fault F-1, was underlain in the near-subsurface by the plane of the fault.

The analysis was then shifted downstream to the C-3 alignment. Although this alignment also presents numerous geologic problems, it was felt that it's use resulted in a more conservative estimate of the overall cost and design effort associated with a gravity dam at the RM 20.1 site. The C-3 alignment is situated so that the left abutment also overlies the subsurface trace of Fault F-1, but due to the dip of the fault plane, it lies at a significantly greater depth than under the C-1 alignment. A disadvantage of the C-3 alignment is the location of a major landslide (Slide 16) which underlies or is immediately downstream of the foundation footprint of the dam. Investigations conducted by the USBR indicate the volume of the slide exceeds 1-million cubic yards of material. Due to the geometry of the slide and it's proximity to the dam foundation, it was determined that complete removal of the slide will be necessary. The final alignment for the selected plan will be reexamined and analyzed in more detail during PED studies.

Future Facilities

Provisions for potential future power facilities were considered for inclusion in the selected plan. However, the local sponsor after evaluation has requested that no future facilities be included. It appears that it is more economical to construct these facilities in the future when and if the need for them occurs.

Shape of Dam Cross Section

Initial analysis of the alternatives for concrete dams at the damsite proposed a traditional shape for a flood control only dam and a trapezoidal shape for a dam that could be made larger in the future. The traditional shape would have a vertical downstream face for a distance of 30-40 feet and would transition into a sloping downstream face, see Figure 8. This shape conserves concrete in an area of the dam where concrete mass is not normally needed. The trapezoidal shape would begin the sloping downstream face immediately from the top, see Figure 9. This continuous sloping face would aid bonding and load transfer of any new concrete placed on the old concrete should the dam ever be enlarged.

A more detailed seismic analysis has shown that high stress

concentrations would occur in the area of transition from vertical to sloping face in the traditional shape dam during a seismic event. These stress concentrations do not occur in a trapezoidal shape dam, compare Figures 8 and 9. The stresses in the traditional shape are two to three times higher than in the trapezoidal shape. Because this dam will be constructed in a seismic area, the decision has been made to design the concrete flood control structure at RM 20.1 with a trapezoidal shape to minimize these stress concentrations. In addition, RCC placement methods are easier to accomplish with a sloping face. This trapezoidal shape, as shown on Plate 9, is the one that would be used for any structure at this site without regard to any future enlargement.

Relocations

The relocations for the selected plan include the relocation of California State Highway 49, replacement of the Ponderosa Way Bridge, and the relocation of telephone and power lines. Description of the work required for these relocations is the same as given earlier in this chapter for the dam alternatives.

Real Estate Requirements

Lands required for the dam include the lands under the dam itself as well as spoil disposal areas which will be acquired in fee; the lands required for the relocation of Highway 49 and the replacement of Ponderosa Way Bridge which will be acquired as road easements; and the lands behind the dam subject to periodic inundation which will have only easements purchased. Relocation of the utilities will be placed within the right of way of the Highway 49 relocation and additional lands are not necessary for these. Easements will also be acquired for several temporary construction staging areas. The road easements are also within the area of the flood easements. Table N-3-21 gives approximate acres necessary for the selected plan.

TABLE N-3-21

SELECTED PLAN
REAL ESTATE REQUIREMENTS
DAM PORTION

Dam, Waste and Staging Areas	100 Acres - Fee
Highway 49 Relocation	52 Acres - Easement
Flood Inundation Lands	5,932 Acres - Easement

Site Geology

Although no explorations have been conducted by Sacramento District for the proposed flood control dam, extensive geologic mapping of the entire area by United States Bureau of Reclamation (USBR) personnel and the close proximity of the extensive explorations completed for the thin-arch dam and the alternative dams provides information for assessment of the geologic conditions at the proposed damsite. The current COE alignment is located approximately 800 feet downstream of the centerline of the alignment of the thin-arch dam. Mapping done by the USBR indicate that their alignments and that of the proposed COE alignment lie along the general northwest strike of many of the structural and lithologic features within the bedrock in Auburn Dam area, and thus many of the geologic features found in the foundation of the thin-arch dam can be projected into the foundation of the COE damsite.

Much of the geologic information presented in this discussion was taken from USBR reports titled "Auburn Dam Excavation and Treatment, Records of Geologic Investigations, Part I of IV", "Project Geology Report, Seismic Evaluation of Auburn Damsite, Volume 1 of 3", and "Design and Analysis of Auburn Dam, Volume One", as well as unpublished maps and personal communications with current and former USBR Auburn Dam project geologists.

<u>**Surficial Units**</u> - Much of the surface area is covered with some type of surficial material; colluvium, landslides, river alluvium, or material deposited in the river channel when the upstream coffer dam failed in 1986. The largest exposure of rock occurs in portions of the river channel and parts of the channel wall excavated to rock during foundation and spillway stripping for the thin-arch dam.

The major component of colluvium is slopewash, which according to the USBR reports ranges from 0.5 to 10 feet thick, averages about 2 feet thick, and consists of weathered rock fragments in a loose matrix of silty soil. This material forms on slopes subject to mass wasting and generally overlies 1 to 4 feet of rock affected by creep.

Units designated as landslide are very old rubble slides that in most cases have attained equilibrium, but in some cases have been reactivated by undercutting during excavation associated with the thin-arch dam.

Over thirty small to large landslides occurred during construction of access roads and foundation excavation for the thin-arch dam. The largest landslide stabilized during construction, designated slide 21 by the USBR, was located on the canyon wall which will form the left abutment of the COE dam and contained an estimated volume of 400,000 cubic yards (CY). Removal of the slide and resloping of the underlying weathered bedrock required excavation of 1,080,000 CY of material. That material was placed as engineered fill in three ravines downstream of the left abutment of the proposed COE dam (See Surface Geology Map, Plate 10). (NOTE: Plate 10 is too large to include in this appendix.

It will be sent in a separate package to reviewers. Others who wish to view this plate may do so at the Geology Section of the Sacramento District.)

A large block slide (Slide 16, Surface Geology Map) underlies and extends downstream of right abutment of the foundation of the COE dam. Investigations conducted by the USBR indicate that the maximum depth of the slide is approximately 100 feet and that it contains approximately 1,200,000 CY of material. Slope indicators (DH-358 and DH-361, Surface Geology Map) were installed on the slide in 1979 by the USBR to monitor the down-slope movement of the slide. The casing in those holes sheared off at depths of approximately 83 and 95 feet. Because of it's blocky structure and the highly foliated metasedimentary rock within, and adjacent to the slide, it's downstream (southeasterly) extent is not known, but is believed to be approximately 700-800 feet wide. Due to the geometry of the slide and its proximity to the dam foundation, it is anticipated that complete removal of the slide will be necessary prior to construction. It is estimated that removal of the slide and resloping of the channel wall will require excavation of more than 1,500,000 CY of material.

Another slide which may be of possible concern is located downstream of the alignment, opposite the outlet portal for the diversion tunnel (Slide 27, Surface Geology Map). When the coffer dam failed in 1986 the slide was partially washed away by the extremely high flows caused by the rapid release of the reservoir behind the dam. Additional explorations will be required to determine the extent to which the slide will be impacted by future flooding, and whether consideration should be given to constructing a structure to protect the slide. An alternative would be to leave the slide in place and deal with it on an as needed basis. In the event of flooding great enough to impact the slide, any slide material would be carried downstream and away from the diversion tunnel.

As a consequence of the 1986 coffer dam failure, extensive quantities of gravel, cobble, and boulder-size fragments of material were deposited in the river channel downstream of the coffer dam. In the vicinity of the proposed COE dam alignment, as much as 85 feet of the material was deposited over bare rock. Although no gradational testing of the materials have been conducted, it is suspected that the sandy portion of the embankment was carried down stream by the flood waters and is absent in the channel deposits underlying the COE alignment.

Rock Units - Amphibolite is the predominant and most competent rock type at the site with a elastic modulus of 14 million psi. It is a hard, dense, fine-grained feldspar-amphibole schist, containing variable amounts of chlorite, epidote, carbonate, and quartz. Texturally, it ranges from highly schistose to subschistose. Based on information developed by the USBR, the foliation is relatively uniform throughout the area, with an average strike of about N30°W and an average dip of about 80°NE. K-Ar dates on amphibolite from the area suggest a

metamorphic age of about 190 million years B.P. Mapping conducted by the USBR indicates that approximately 80-85% of the proposed COE dam will be underlain by amphibolite.

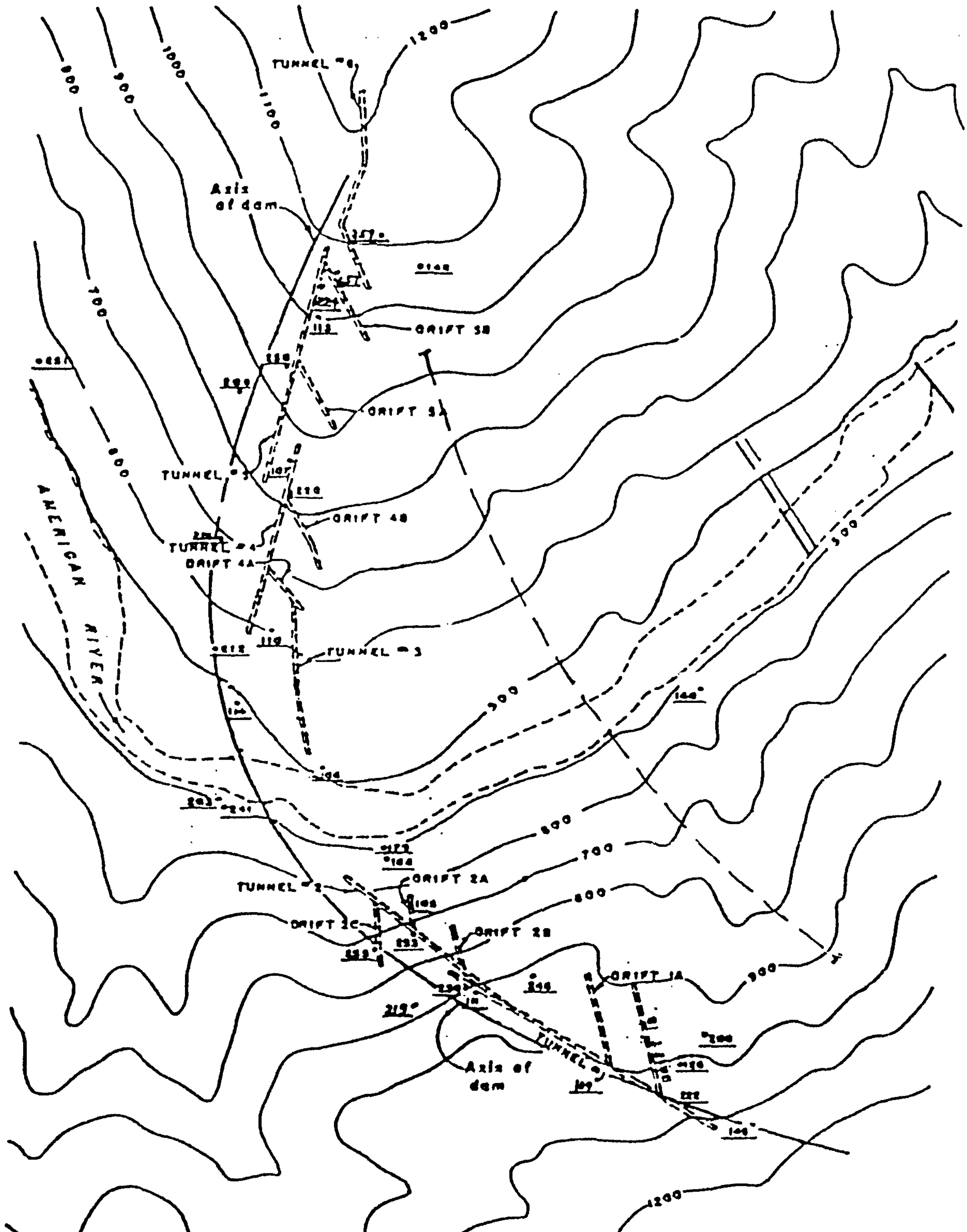
Metasediments is a collective term for a variety of metamorphosed sedimentary rocks that occur locally as thin interbeds within the amphibolite. Laminated metashale and thin-bedded metasandstone, which are slightly softer and less resistant to weathering than the amphibolite, are the major varieties. Harder, highly siliceous metachert and quartzite occur locally.

Metagabbro and gneissic metagabbro occur locally and have a greater and coarser feldspar content than amphibolite. The metamorphic structure of these rocks ranges from foliated to massive but rock properties are otherwise similar to amphibolite.

Serpentine occurs downstream of the right abutment of the proposed COE damsite opposite the downstream portal of the diversion tunnel, and locally as small isolated pods and lenses. The serpentine is typically massive, hard, and moderately fractured.

Talcose and chloritic rocks, some of which are closely associated with serpentine, are of special importance because they are the least competent rocks in the proposed dam foundation. Individual bodies or complexes of these units, which have been designated as T-zones will comprise approximately 15% of the rock in the foundation (see Surface Geology Map). The USBR has subdivided these rocks in decreasing order of strength, as chlorite schist C, talcose serpentine, chlorite schist B, chlorite schist A, and talc schist. The T-zones occur mostly as tabular zones parallel to the metamorphic foliation and occasionally as discordant masses which are largely composed of serpentine. In the site area, T-zones range in thickness from 0.1-foot to about 200 feet (T-0 high above the left abutment), but most are 1 to 20 feet thick. Generally, the T-zones are discontinuous and commonly bifurcate and braid through the country rock.

<u>Laboratory Tests</u> - During the extensive investigations carried out by the USBR for the Auburn Dam Project, a series of some 530 NX core samples (nominal 2.125-inch diameter) were tested to determine the physical properties of the various rock types. Additionally three talcose serpentine specimens, 6 inches in diameter were also tested. These NX samples were taken from 32 surface drill holes and from 73 underground holes drilled in the various drifts and tunnels. Of these underground holes, 32 were drilled as uniaxial sites with one vertical hole upward through the crown and an opposing vertical hole downward through the invert. Twenty seven holes were drilled as radial sites where the holes are perpendicular to the axis of the drift or tunnel and spaced at 45 degree intervals. The remaining 14 holes were drilled as individual sites in the various drifts and tunnels. Samples from several different depth intervals were taken from each hole. As expected, the USBR concentrated their testing efforts along the proposed axis of their dam. The holes, drifts, and tunnels from which the samples were taken are shown on Figure 17.



Location map of drill holes and tunnels - Auburn Dam.

Dashed line represents approximate COE alignment.

FIGURE 17

Although the axis of the COE dam is a short distance downstream from the USBR axis, the physical properties of the various rock types should remain valid for the COE site. The dashed line on Figure 17 is the approximate COE axis.

The core samples were classified into 7 rock types: amphibolite, massive amphibolite, foliated amphibolite, metagabbro, chlorite schist, talc schist, and dike. There were minor variations in each of these rock types plus lesser amounts of gneissic metagabbro, metasediments, quartzite, serpentine, and talcose serpentine. The physical properties tested for include the following: specific gravity, absorption, porosity, secant modulus of elasticity, Poisson's ratio, compressive and triaxial strengths, and sonic velocity. In general the moduli of elasticity ranged from 1,470,000 lb/in^2 to 17,310,000 lb/in^2: Poisson's ratio ranged from 0.11 to 0.24; compressive strengths ranged from a low of 790 lb/in^2 to a high of 41,900 lb/in^2; and sonic velocities ranged from 12,950 ft/sec to 29,900 ft/sec.

Over 110 core samples were tested in direct shear and over 120 samples were tested in triaxial shear. The results listed below summarize average values for these tests.

Rock type	Triaxial shear		Direct shear	
	Cohesion lb/in^2	Tan Phi	Cohesion lb/in^2	Tan Phi
Amphibolite	2,750	1.33	2,200	1.96
Amphibolite (with healed joints)			600	2.50
Metagabbro	2,450	1.43		
Talc schist	1,300	0.34		
Chlorite schist	1,500	0.78	2,030	1.40
Metasediments	2,500	1.48	2,400	1.48
Serpentine			2,150	0.97

Additional information on rock properties was obtained by running a series of in situ uniaxial jacking tests, plate gouge tests, shear tests, and borehole stress relief tests in various drift and tunnel locations.

Tables N-3-22 and N-3-23 present a summary of the physical properties of the tested core samples by rock type. A complete description of the physical properties of each piece of core tested is contained in the report, "Laboratory Tests of Foundation Rock Core for Auburn Dam, GR-12-75, October 1975" and is on file in Geology Section, Sacramento District.

TABLE N-3-22

SPECIFIC GRAVITY, ABSORPTION, AND POROSITY BY ROCK TYPE

Rock Type	No of samples	Bulk Specific Gravity	Absorption % by wt.	Porosity % by vol.
Amphibolite	313	2.65-3.13 avg 2.98	0.00-3.08 avg 0.12	0.00-8.56 avg 0.36
Massive Amphibolite	11	2.92-3.07 avg 3.03	0.01-0.07 avg 0.03	0.03-0.21 avg 0.09
Foliated Amphibolite	10	2.93-3.04 avg 2.99	0.02-0.11 avg 0.06	0.06-0.32 avg 0.18
Metagabbro	42	2.90-3.11 avg 2.98	0.00-0.47 avg 0.08	0.00-1.40 avg 0.23
Chlorite Schist	34	2.77-2.95 avg 2.85	0.00-0.94 avg 0.26	0.00-2.62 avg 0.72
Talc Schist	16	2.76-2.92 avg 2.85	0.00-0.78 avg 0.29	0.00-2.21 avg 0.82
Dike	11	2.74-2.90 avg 2.79	0.02-0.52 avg 0.13	0.06-1.43 avg 0.36

TABLE N-3-23

MODULUS, POISSON'S RATIO, COMPRESSIVE STRENGTH, AND SONIC VELOCITY BY ROCK TYPE

Rock Type	Number Range Avg	Modulus of Elasticity 0-5000 lb/in² 2nd cycle E x 10^6	Poisson's Ratio 0-5000 lb/in² 2nd cycle	Compressive Strength, lb/in²	Sonic Velocity ft/sec
Amphibolite	Number	228	216	314	194
	Range	4.63-21.37	0.10-0.30	620-51,700	12.9K-29.9K
	Avg	13.45	0.22	18,820	23,580
Massive Amphibolite	Number	6	6	9	6
	Range	12.69-16.61	0.12-0.27	8550-28,660	25.0K-27.2K
	Avg	14.85	0.22	20,790	26,040
Foliated Amphibolite	Number	3	3	9	-
	Range	15.15-17.24	0.21-0.26	11.1K-31.2K	-
	Avg	16.45	0.23	20,890	-
Metagabbro	Number	29	27	41	23
	Range	6.35-16.23	0.18-0.30	3520-41,900	21.3K-28.3K
	Avg	14.37	0.23	24,310	25,350
Chlorite Schist	Number	7	7	32	19
	Range	7.70-16.34	0.19-0.27	1430-16,370	13.6K-26.3K
	Avg	10.67	0.24	5,980	20,160
Talc Schist	Number	* 13	* 5	13	11
	Range	1.13-6.49	0.12-0.25	790-4,510	18.4K-22.8K
	Avg	3.22	0.19	2,630	20,040
Dike	Number	4	3	11	-
	Range	7.28-10.57	0.21-0.26	3600-30,840	-
	Avg	8.66	0.23	17,740	-

* The only tests run were at 0-1000 lb/in², on the 2nd cycle.

<u>Permeability Tests</u> - In addition to obtaining core samples from all of the holes drilled during the foundation investigations, the holes were all water tested to provide information on foundation permeability and also to gather data for probable grout takes. Standard permeability tests were run in over 350 drill holes and a program of 119 special

tests were run in over 350 drill holes and a program of 119 special permeability tests was conducted in an additional 85 drill holes located in various drifts, tunnels, and at the surface. These special permeability tests were formulated to give data on specific faults and shears, as well as on the bulk rock. The tests were performed utilizing special equipment after the drill hole had been cleaned and evacuated of standing water. The results of these tests gave average coefficients of permeability (K) which ranged from less than 1.0 ft/yr to 638 ft/yr. In general these tests confirmed what had been indicated earlier in the exploration program, that the foundation rock has very low permeability. For example, a series of nine right abutment holes had average coefficients of permeability values generally less than five feet per year for test lengths of 10 to 20 feet. Representative values of K obtained through this program are shown in Table N-3-24.

The USBR also formulated another test called an "exit gradient test". This test was set up to detect possible piping along faults, shears, and talc zones by using an AX-size hole drilled normal to and through the fault or shear to be tested. Water colored with dye was injected into the hole at incremental pressures up to 260 lb/in^2. The pressure was held at each step for about 20 minutes with the top pressure maintained for about 10 hours. Flow was observed in less than 45 percent of the tests with piping in less than 4 percent. The results of the "exit gradient tests" with the calculated permeabilities are shown in Table N-3-25.

__Structural Units__ - The two major structural features in the dam foundation are the T-zones (discussed above) which parallel the metamorphic foliation, and F-zones which crosscut the foliation. Since the damsite lies between branches of the Bear Mountains fault zone, some of the serpentine-related T-zones in the foundation might be considered small splays off these branches. The T-zones are lithologic as well as structural units and are not necessarily related to Mesozoic faulting.

F-zones are crosscutting faults which have been delineated over a significant portion of the USBR's study area. The majority of these faults within the damsite area dip moderately to the southwest. They consist of one or more shear zones composed of highly variable amounts of clay gouge, rock fragments, and quartz-calcite veinlets. Typically, dikes have intruded along these faults, suggesting they formed 130 to 140 million years B.P. in response to the intrusion of the Penryn and Rocklin plutons a few miles southwest of the site.

As shown on the Surface Geology Map (Plate 10), few faults identified by the USBR in their investigation for the thin-arch dam can be projected into the foundation of the proposed COE dam with reasonable confidence. Those faults which are expected to be encountered in the foundation of the COE dam include F-3, F-10, F-16, F-17, and F-25, as well as any number of previously unmapped faults.

TABLE N-3-24

AUBURN DAM

Permeability Data Transmitted by October 5, 1973

Discon-tinuity	D. H. No.	Interval ft	L ft	Drill size	C	Water loss gpm 25	50	100	150	Pressure psi 25	50	100	150	Pressure ft 25	50	100	150	K ft per yr 25	50	100	150	K av ft per yr	Remarks
F-10	D. H. 2-20	35.8 to 38.8	3	NX	11846	0	0.01	0	0	34	55	106	124	78.5	127	244.6	286	0	1.0	0	0	0.3	R. J. Before Jetting)
T-14, F-2	(TU2)	117.8 to 217.9	100.1		747				0.12				10				23				4	4.0	R. J.
T-14, F-2	El 523.9	118.8 to 217.9	99.1		753				0.03				10				23				1	1.0	R. J.
T-14, F-2	Dip = -1°	111.8 to 217.9	106.1		711.5				0.03				10				23				1	1.0	R. J.
	Block I																						
	See F''-F'''			NX																			
F-8	D. H. 2-22	136.3 to 144.8	8	NX	3813	0.18	0.46	1.13	1.38	10	34	83	104	23	78	192	240	45.5	34.3	34.2	33.4	37	R. J. *0.1 gpm backflow
Healed breccia	(TU2)	173.5 to 181.5	8		3813	1.24	5.86	10.9	11	17	41	86	104	39	95	198	240	184.8	358.6	293.6	266.4	276	R. J. No back pressure taken for tests 2, 3, and 4, and no back pressure was applied to test result. It is possible up to 20 psi back pressure was present.
T-14, F-2	El 524.3	232 to 279.2	47.2		1406	0.02	0.14	0.33	0.36	31	54	106	124	72	125	245	286	0.4	1.6	1.9	1.8	1.4	
T-14	Dip = -10°	241.2 to 279.2	38		1682	*0	0.1	0.3	-	32	56	105		74	129	242	-	0	1.3	2.1	-	1.1	
	Blocks I, J																						
	See F''-F'''			NX																			
Block J	D. H. 218	427.5 to 447.4	19.9	NX	2849	0.26	0.32	0.62		66	91	144		152	210	332		4.9	4.3	5.3		4.8	A. J. (After Jetting)
Block J	Rt abut	411.7 to 431.7	20		2838	0.12	0.18	0.28		64	87	141		148	201	325		2.3	2.5	2.4		2.4	A. J.
S-23	El 985	392.7 to 412.7				0.1	0.24	0.4		62	88	141		143	203	325		2.0	3.4	3.5		3.0	A. J.
Block J	Dip = -85°	373.6 to 393.6				0.42	0.56	0.85		70	96	144		162	222	332		7.4	7.2	7.3		7.3	A. J.
Block J	Block J	354.5 to 374.5				0.16	0.3	0.46		57	87	139		132	201	321		3.4	4.2	4.1		3.9	A. J.
S-22	See FF''-F'''	336.3 to 356.3				0.16	0.28	0.44		63	90	142		145	208	328		3.1	3.8	3.8		3.6	A. J.
F-13, T-18	See UU-UU'	317.3 to 337.3				0.34	0.48	0.70		72	96	146		166	222	337		5.8	6.1	5.9		5.9	A. J.
Block J		298.2 to 318.2				0.22	0.32	0.44		72	96	147		166	222	339		3.8	4.1	3.7		3.9	A. J.
S-21		279.3 to 299.3				0.24	0.28	0.44		75	96	145		173	222	335		3.9	3.6	3.7		3.7	A. J.
Block J		260 to 280				0	0	0.02		70	90	145		162	208	335		0	0	0.4		0.1	A. J.
S-20		241.4 to 261.4				0.02	0.04	0.08		73	96	146		168	222	337		0.3	0.3	0.7		0.4	A. J.
Block J		220.3 to 240.3				0.18	0.18	0.38		74	98	148		171	226	341		3.0	2.3	3.2		2.7	A. J.
T-18		201.5 to 221.5				0	0.04	0.1		70	96	147		162	222	339		0	0.3	0.8		0.4	A. J.
T-18, S-13		**165 to 185	20	NX	2838	0.02	0.02	0.04		74	98	146		171	226	337		0.3	0.3	0.3		0.3	A. J. **12' † from wall of drift 1 A
Block I	D. H. 226	561.6 to 581.6	20	NX	2838	0.04	0.06	0.13		67	97	***		155	224	692+		0.7	0.8	0.5		0.6	A. J.
Block I	Rt abut	547.4 to 567.4				0.2	0.23	0.35		80	105	***		185	242	692		3.1	2.7	1.4		2.4	A. J.
Block I	El 694.8	528.2 to 548.2				0.08	0.12	0.20		69	100	***		159	231	692		1.4	1.5	0.8		1.2	A. J.
S-33, S-32	Dip = -83°	509.2 to 529.2				0.1	0.12	0.20		81	106	***		187	245	692+		1.5	1.4	0.8		1.2	A. J.
Block I	Block I	490.1 to 510.1				0.14	0.15	0.23		82	106	159		189	245	367		2.1	1.7	1.8		1.9	A. J.
Block I	See PP-PP'	471.4 to 491.4				0.11	0.12	0.18		83	107	160		192	247	369		1.6	1.4	1.4		1.5	A. J.
S-31, S-32	See C-C'	452.5 to 472.5				0.01	0.1	0.15		72	96	153		166	222	353		0.2	1.3	1.2		0.9	A. J. ***Piezometer pressure exceeded the readout gauge
S-29, 30, 31		433.2 to 453.2				0.07	0.1	0.16		83	107	158		192	247	365		1.0	1.1	1.2		1.1	A. J. (300 psi)
S-27, 28		414.2 to 434.2				0.7	0.09	0.16		87	109	161		201	252	372		9.9	1.0	1.2		4.0	A. J.
Block I		396.0 to 416				0.14	0.18	0.23		85	111	158		196	256	365		2.0	2.0	1.9		2.0	A. J.
Block I		376.7 to 396.7				0.06	0.07	0.11		87	114	161		201	263	372		0.8	0.8	0.8		0.8	A. J.
Block I		376.7 to 396.7				0	0	0.08		87	113	163		201	261	376		0	0	0.6		0.2	A. J.
Block I		357.6 to 377.6				0.06	0.08	0.1		86	111	159		198	256	367		0.9	0.9	0.8		0.9	A. J.
S-26		338.6 to 358.6	20	NX	2838	0.38	0.46	0.58		86	112	161		198	258	372		5.4	5.1	4.4		5.0	A. J.

TABLE N-3-24 (CONT.)

AUBURN DAM
continued
Permeability Data Transmitted by October 5, 1973

Discontinuity	D. H. No.	Interval ft	L ft	Drill size	C	Water loss gpm 25	Water loss gpm 50	Water loss gpm 100	Water loss gpm 150	Pressure psi 25	Pressure psi 50	Pressure psi 100	Pressure psi 150	Pressure ft 25	Pressure ft 50	Pressure ft 100	Pressure ft 150	K ft per yr 25	K ft per yr 50	K ft per yr 100	K ft per yr 150	K av ft per yr	Remarks
Block K3	D. H. 255 Rt abut El 1,049.7 Dip = -43° Sec TT-TT' Block K	420 to 440	20	NX	2838	4.27 3.9	6.34 9.7	9.6		30 36	57 58	104		69 81	132 134	240		175.6 133.4	136.3 205.4	113.5		153	R. I. (Before Jetting)
S-13		400 to 420				4.0 3.0	5.8 4.9	8.0		46 49	75 76	122		106 113	173 175	282		155.3 123.1	95.8 79.5	80.5		107	A. I. (After Jetting)
F-18		380 to 400				1.1	1.6	2.4		47	75	124		108	173	286		28.9	26.2	23.8		26	R. J.
Block		360 to 380				0.67	0.72	1.15		56	84	131		129	194	302		14.7	10.5	10.8		12	A. J.
Block K3		340 to 360				0.09	0.2	0.38		49	75	127		113	173	293		2.3	3.3	3.7		3	R. J.
S-10		320 to 340				0	0	0.01		-	-	131		-	-	302		0	0	0.1		0	A. I.
S-8, 9		300 to 320				0.80	1.49	2.57		51	81	130		118	187	300		19.2	22.6	24.3		22	A. J.
Block K3		280 to 300				0	0	0		-	-	-		-	-	-		-	-	-		0	A. J.
S-7		260 to 280				3.18	7.79	16.4		55	72	120		127	166	277		58.2	133.2	168.0		120	A. J.
S-5, 6		240 to 260				.19	0.84	0.92		56	80	133		129	185	307		6.2	12.9	8.5		8.5	A. J.
Block K3		220 to 240				0.33	0.59	0.87		55	81	132		127	187	305		7.3	9.0	8.1		8.1	A. J.
S-3, 4		200 to 220				0.06	0.35	0.81		55	78	134		127	180	309		1.3	5.5	7.4		4.7	A. I.
S-3		180 to 200				0.02	0.02	0.02		57	80	132		132	185	305		0.4	0.3	0.2		0.3	A. J.
Block K3		160 to 180				1.18	1.56	2.47		57	79	133		132	182	307		25.4	24.3	22.8		24	A. J.
F-20		135 to 155				0.69	1.13	2.06		57	78	130		132	180	300		14.8	17.8	19.5		17	A. J.
F-20		120 to 140				2.16	3.35	6.95		53	79	129		122	182	298		50.2	52.2	66.2		56	A. J.
Block K3		100 to 120				2.45	3.4	5.35		57	82	129		132	189	298		52.7	51.0	60.0		55	A. J.
Block K3		80 to 100				2.43	3.95	-		56	80	-		129	185	-		53.5	60.6	-		57	A. J.
Block K3		60 to 80	20		2838	2.11 5.95	3.35 9.4	13.5		55 53	84 76	121		127 122	194 175	279		47.2 138.4	49.0 152.4	137.3		105	A. J.
S-13		411.2 to 414.2	3		11846	3.97 2.9	5.47 4.27	7.33		54 56	76	124		125 129	175	286		376.2 266.3	370.3 289.0	303.6		321	A. J.
F-20		136.9 to 139.9	3		11846	0.73	1.03	1.66		57	78	130		132	180	300		65.5	67.8	65.5		66	A. J.
F-18		382 to 392	10	NX	4900	1.39	1.79	2.33		54	82	130		125	189	300		54.5	46.4	41.3		47	A. J.
F-10	D. H. 204 Rt abut El 888.9 Dip = -88° Sec [illegible] Block [illegible]	473.1 to 492.9	19.8	NX	2861	0.2	0.2	0.3		187	210	263		432	485	607		1.3	1.2	1.4		1.3	A. J.
F-3, 10		454.3 to 474.1				35.3 15.2	25.8 22.7	34.5		*125 171	178 183	202		*288 395	411 422	466		*350.7 110.1	179.6 153.8	211.8		201	A1 *Data questionable; test water leaking around packer intervals and some test water entering from D. H. 2 22 in [illegible]. Open conduits apparently [illegible] from 169' to 460.0' and [illegible].
R-23		438.2 to 458.0				20.1 9.7	13.6 11.7	15.4		182 179	201 201	269		420 411	468 464	575		68.8 61.7	70.9 69.1	76.6		70	A1
S-23		419.6 to 439.4				22.1 14.2	21.5 17.8	25.6		163 178	189 195	228		376 411	435 450	326		169.7 98.8	161.1 113.2	139.2		132	A. J.
Block I4		403.0 to 422.8				9.47 9.13	11.7 11.5	17.1		185 183	205 204	247		427 422	473 471	570		63.5 61.9	70.8 69.8	85.8		70	A. J.
Block I4		383.8 to 403.6	19.8		2861	0.87	0.98	1.2		186	211	260		429	487	600		5.8	5.8	5.7		5.8	A. J.
S-24		365.7 to 384.3	18.8		2983	18.3 12.6	18.6 14.6	23.4		169 178	195 199	235		390 411	450 459	542		140.0 91.4	123.3 94.9	128.8		116	A. J.
Block I4		345.7 to 365.5	19.8		2861	0	0	0.4		-	-	156		-	-	360		0	0	3.2		1.0	Std (Standard Tests)
S-22, 23		326.5 to 346.3	19.8		2861	0	0	0.2		-	-	156		-	-	360		0	0	1.6		0.5	Std
F-11		307.3 to 327.1	19.8		2861	2.83	3.4	5.8		180	205	254		415	473	586		19.5	20.6	28.3		26	A. J.
Block I3		288.9 to 308.7	19.8		2861	0.03	0.05	0.13		171	196	248		395	452	572		0.2	0.3	0.7		0.4	A. J.
Block I3		291.6 to 311.6	20		2838	0	0	0.6		-	-	156		-	-	360		0	0	4.7		1.6	Std
Block I3		278.8 to 298.6	19.8		2861	0	0	0		-	-	-		-	-	-		-	-	-		0	A. I.
Block I3		278.3 to 293.8	15		3569	0	0	0.4		-	-	156		-	-	360		0	0	4.0		1.3	Std
S-18		254.8 to 274.6	19.8	NX	2861	0	0.4	0.8		-	106	156		-	245	360		0	4.7	6.3		1.7	Std

TABLE N-3-24 (CONT.)

AUBURN DAM
continued
Permeability Data Transmitted by October 5, 1973

Discon-tinuity	D. H. No.	Interval ft	L ft	Drill size	C	Water loss gpm 25	50	100	150	Pressure psi 25	50	100	150	Pressure ft 25	50	100	150	K ft per yr 25	50	100	150	K av ft per yr	Remarks
Block I	D. H. 226	319.5 to 339.5	20	NX	2838	0.8	1.15	1.32		86	111	163		203	256	376		11.2	12.7	10.0		11.3	A. J. (After Jetting)
Block I	Rt abut	319.5 to 339.5	20		2838	0.48	0.5	0.63		88	111	163		203	256	376		6.7	5.5	4.8		5.7	A. J.
F-7	El 694.8 Dip = -83°	300.2 to 320.2	20		2838	0.28	0.3	0.41		88	111	161		203	256	372		3.9	3.3	3.1		3.6	A. J.
Block I	Block I Sec PP-PP' Sec C-C'	291.5 to 301.5	10		4900	0.02	0.02	0.07		90	113	160		208	261	369		0.5	0.4	0.9		0.6	A. J.
S-23, 24		272.2 to 292.2	20		2838	0	0	0		89	113	162		205	261	374		0	0	0		0	A. J.
Block I		253.9 to 273.9				1.1	1.12	0.75		89	114	163		205	263	376		15.2	12.1	5.7		11.0	A. J.
S-21, -22		235.1 to 255.1				0.08	0.10	0.22		87	112	162		201	258	374		0.8	1.1	1.7		1.2	A. J.
S-19, 20		217.8 to 237.8				2.06	2.66	4.3		88	112	163		203	258	376		28.8	29.0	32.5		30	A. J.
S-18		198.9 to 218.9				0.38	0.35	0.6		89	114	161		205	263	372		5.3	3.8	3.1		4.1	A. J.
S-16, 17, 18		179.9 to 199.9				0.26	0.44	1.05		89	112	160		205	258	369		5.3	4.8	8.1		5.4	A. J.
Block I		160.6 to 180.6	20		2838	0	0.01	0.01		86	113	161		198	261	372		0	0.1	0.1		0.1	A. J.
S-15, Vx		151.5 to 161.5	10	NX	4900	9.0 10.4	11.3 12.1	15.6		86	111	154		196	256	355		149 129.0	134.1 125.3	126.7		132	A. J. (Moderate Vx cemented before tests)
		106 to 161.5	cemented before test																				
F-17a, S-28	D. H. 252	396 to 405.3	9.3	NX	5189	0.48	0.51	0.72		129	154	206		298	355	471		8.4	7.5	7.9		7.7	B. J. (Before Jetting)
F-17, 17a S-25, 28	Rt abut El 1,049.7 Dip = -69° Block F	385.3 to 405.3	20		2838	7.5 7.0	8.5 8.0	10.5		154 155	179 178	225		355 358	413 411	519		60.0 55.5	58.4 55.2	57.4		57	Std (Standard Tests)
F-17	Sec II-II'	184.5 to 193.5	9		5314	5.1 4.4	5.8 5.1	7.1		138	158	197		193	340	478		84.9 74.7	85.8 78.2	78.8		82	B. J.
F-18		360.5 to 369.5	9		5314	0	0.02	0.03		119	146	201		275	337	464		0	0.3	0.3		0.2	A. J.
F-18, 23 S-15, 16		348.6 to 368.6	20		2838	0	0.2	0.6		160	185	235		369	427	542		0	1.3	3.1		1.5	Std
F-23, S-15		346 to 355	9		5314	0.54	0.64	0.77		136	158	206		314	365	475		10.8	9.3	8.6		9.6	B. J.
F-24		295.5 to 304.5	9		5314	32.1 25.0	31.7 28.3	34.2		89	115	*156		205	265	360		832.1 648.0	635.7 567.5	504.8		638	B. J. *Maximum collar pressure = 85 psi
F-24		292.5 to 312.5	20	NX	2838	3.0	7.0	16.0		155	177	213		358	408	492		23.8	48.7	92.3		55	Std
F-26	D. H. 2-40 (TU2) El 518.8	73 to 101.7	28.7	NX	2119	0.01	0	0.03 0		30	54	101		69	-	233		0.3	0	0.3 0		0.2	B. J.
F-26	Dip = -90° Block I Sec QQ-QQ'	67.4 to 87.4	20		2838	0.2 0.2	0.4 0.4	0.5		27	52	102		62	120	235		9.1 9.1	9.5 9.5	6.0		8.6	Std
F-26		85.4 to 105.4	20		2838	0	0	0		-	-	-		-	-	-		0	0	0		0	Std
S-4		66.8 to 72	5.2		8063	0.28	0.52	0.95 1.05		25	49	98 118		58	113	226 272		38.8	37.0	33.8 31.0		35	B. J.
S-4		67.4 to 87.4	20		2838	0.2 0.2	0.4 0.4	0.5		27	52	102		62	120	235		9.1 9.1	9.5 9.5	6.0		8.6	Std
S-3		58.7 to 63.9	5.2		8063	0	0	0		-	-	-		-	-	-		0	0	0 0		0	B. J.
S-3		49.9 to 69.9	20	NX	2838	0	0.4	0.9		27	52	102		62	120	235		0	9.5	10.9		6.8	Std
F-3	D. H. 219 Rt abut	267.5 to 274.2	6.7	NX	6666	0.4	0.07	0.16		74	104	157		171	240	362		15.6	1.9	2.9		6.8	B. J.
F-3	El 869.4 Dip = -88°	261 to 281	20		2838	0	0.5	0.5		67	92	142		155	212	328		0	6.7	4.3		3.7	Std
F-8	Sec QQ-QQ'	254 to 260.6	66		6744	0	0	0		-	-	-		-	-	-		-	-	-		0	B. J.
F-8		242.1 to 262.1	20		2838	0	0	0		-	-	-		-	-	-		-	-	-		0	Std
F-10		194.7 to 201.3	6.6		6744	0	0	0		-	-	-		-	-	-		-	-	-		0	B. J.
F-10		185.5 to 205.5	20		2838	0	0	0		-	-	-		-	-	-		-	-	-		0	Std
F-26		360.7 to 378.1	17.4		3174	0.03	0.04	0.09		79	105	153		182	242	353		0.5	0.5	0.8		0.6	B. J.
F-26 F-26		355.2 to 375.2 374.2 to 394.2	20 20	NX NX	2838 2838	0 0	2.2 0	4.0 0.2		- -	91 -	140 141		- -	210 -	323 328		0 0	29.7 0	35.1 1.7		22 0.6	Std Std

TABLE N-3-24 (CONT.)

AUBURN DAM
continued
Permeability Data Transmitted by October 5, 1973

Discon-tinuity	D. H. No.	Interval ft	L ft	Drill size	C	Water loss gpm 25	50	100	150	Pressure psi 25	50	100	150	Pressure ft 25	50	100	150	K ft per yr 25	50	100	150	K av ft per yr	Remarks
S-17	D. H. 204 Rt abut El 888.9 Dip = -88° Sec Q-Q' Block I	235.6 to 255.4	19.8	NX	2861	0.2	0.9	1.4		81	106	156		187	245	360		3.1	10.5	11.1		8.2	Std (Standard Tests)
Block 13		216.6 to 226.4				0	0	0.4		-	-	156		-	-	360		0	0	3.2		1.0	Std
S-16		197.6 to 217.4				0	0.1	0.5		81	106	156		187	245	360		0	1.2	4.0		1.7	Std
F-12, S-15		178.4 to 198.2				0.52	0.47	0.97		105	131	180		242	302	415		6.1	6.3	6.7		6.4	A. J. (After Jetting)
S-12 (T-27a)		159.3 to 179.1				0.6	1.4	2.1		81	106	156		187	245	360		21.4	16.3	16.7		18	Std
S-11 (T-27a)		141.5 to 161.3				0.3	1.0	1.8		81	106	156		187	245	360		4.6	11.7	14.3		10	Std
Block 12		125.9 to 145.7				1.8	3.0	3.9		81	105	154		187	242	355		27.5	35.4	31.4		31	Std
F-13		106.7 to 126.5	19.8		2861	0.4	2.4	4.2		75	100	149		173	231	344		6.6	29.7	34.9		24	Std
F-8		495.4 to 505.8	10.4		4761	0.3	0.5	0.6		81	106	156		187	245	360		7.6	9.7	7.9		8.4	Std
F-13		108.0 to 113.6	5.6		7629	0.68	0.99	2.05		78	102	153		180	235	353		28.8	32.1	44.3		35	A. J.
F-12		180.8 to 191.9	11.1		4522	0.54	0.65	0.95		108	132	182		249	305	420		9.8	9.6	10.2		10	A. J.
F-11		317.8 to 327.6	19.8		2861	3.39 3.39	4.05 4.49	6.92		165 165	188 188	237		381	434	547		25.4 25.4	26.7 29.6	36.2		31	A. J.
F-3		456.1 to 471.3	15.2		3533	28.07 19.3	29.8 23.9	36.3		145 165	167 177	198		335 381	385 408	457		296 179	273 207	281		247	A. J.
F-3, F-10		454.4 to 474.4	20		2838	0.4	1.1	2.4		81	106	154		187	245	355		6.1	12.7	19.2		13	Std
S-25		430.8 to 445.9	15.1		3551	0	0	0		-	-	-		-	-	-		-	-	-		0	A. J.
Block 14		416.0 to 431.1	15.1		3551	17.2 13.3	18.9 16.4	24.1		171 176	192 195	233		395 406	443 450	538		155 116	151 129	159		142	A. J.
Block 14 S-24		419.6 to 439.6	20		2838	0.2	0.3	0.8		81	106	156		187	245	360		3.0	3.5	6.3		4.3	Std
F-3, 8, 10 S-24		**370.2 to 505.8	135.6	NX	576	26.7	22.9	30 ***25.5		154	184	230 269		355	425	531 621		43.3	31.0	32.5 23.7		33	A. J. ***Collar pressure = 145

** Note: 16 individual single packer tests were run in this interval to determine location of open conduit(s) which parallel hole from 369' to 460', allowing test water to move around test intervals. During these tests, D. H. 2-22, TU2, was packed off. ([illegible] pressure read at D. H. [illegible] correlated with water level measurements in D. H. 204.) Results showed open conduit(s) to be steeply dipping cleavage and flatter joints from 369' to 460' in companion with F-3.

TABLE N-3-25

EXIT GRADIENT TEST RESULTS

Discontinuity	Test No.	Location	Hydraulic head (ft)	K ft/yr	Exit gradient head (ft)	Remarks
F-2	1	TU1	600	0.1	600	Report dated 5/1/72, No exit
F-2, T-14	1	TU1	600	0.3	600	Report dated 5/1/72, No exit
F-7	1	TU2	600	30	35	Report dated 5/1/72, Exit
F-7	1	TU2	58	1,600	0	Report dated 5/1/72, Exit
F-13	1	TU1	600	1	600	Report dated 5/1/72, No exit
F-13	1	TU1	600	170	404	Report dated 5/1/72, Exit
F-13	1	TU1	600	4	600	Report dated 5/1/72, No exit
F-17	1	TU1	600	250	58	Report dated 5/1/72, Exit
F-23	1	TU1	600	130	461	Report dated 5/1/72, Exit
F-23B	2	TU1	173	2,300	58	Report dated 5/1/72, Exit
F-24	1	TU1	600	330	115	Report dated 5/1/72, Exit
T-1B	6	Dr. 1A	461	88	346	Report dated 5/1/72, Exit
T-19A, B	4	Dr. 1A	600	7	115	Report dated 5/1/72, Exit
T-19B	1	TU1	600	1	600	Report dated 5/1/72, No exit
T-19B	1	Dr. 1A	461	130	404	Report dated 5/1/72, Exit
T-19C	2	Dr. 1A	461	200	346	Report dated 5/1/72, No exit
T-19C	3	Dr. 1A	461	770	600	Report dated 5/1/72, No exit
T-20	1	Dr. 1A	600	5	600	Report dated 5/1/72, No exit
T-25	1	TU1	115	760	115	Report dated 5/1/72, Exit
Shear 3	5	Dr. 1A	404	320	400	Report dated 5/1/72, Exit
Shear 24	3	TU1	600	37	600	Report dated 5/1/72, No exit
Shear 25	2	TU1	600	48	600	Report dated 5/1/72, No exit
F-1	2	TU4	600	0.1	600	Report dated 5/3/72, No exit
F-1	3	TU4	600	1	600	Report dated 5/3/72, Exit
F-1	1	TU4	600	0.9	600	Report dated 5/3/72, Exit
F-1	1	TU4	600	1.9	600	Report dated 5/3/72, No exit
F-1	2	TU4	600	0.1	600	Report dated 5/3/72, No exit
F-1	3	TU4	600	0.6	600	Report dated 5/3/72, No exit
F-1	1	TU4	600	0.4	600	Report dated 5/3/72, Exit
T-6	1	TU4	600	130	300	Report dated 5/3/72, Exit
T-6	2	TU4	600	0	600	Report dated 5/3/72, No exit
T-6	3	TU4	600	0.1	600	Report dated 5/3/72, No exit
T-6	1	TU4	600	45	460	Report dated 5/3/72, Exit
T-6	2	TU4	600	2.3	346	Report dated 5/3/72, Exit
T-6	3	TU4	600	0.1	600	Report dated 5/3/72, No exit
T-7	1	Dr. 4B	600	0.1	600	Report dated 5/3/72, Exit
T-7	1	Dr. 4B	600	63	519	Report dated 5/3/72, Exit
T-7	1	Dr. 4B	600	0.8	600	Report dated 5/3/72, Exit
T-8A	1	TU4	600	0.2	600	Report dated 5/3/72, No exit
T-3	1	TU5	462	0.3	462	Report dated 3/10/72
T-6a	1	TU4	350	0.5	350	Report dated 9/8/71 (DH-4-70)
T-6	1	Dr. 4B	350	0.5	350	Report dated 9/8/71 (DH-P-4B-5L)

TABLE N-3-25 (CONT.)

Discontinuity	Test No.	Location	Hydraulic head (ft)	K ft/yr	Exit gradient head (ft)	Remarks
T-6	1	TU4	115	66.0		Report dated 9/8/71 (DH-4-70), K shown as variable with head. Results suspect.
F-1	1	TU4	700	0.85	700	Report dated 9/8/71 (DH-P-4-36L)
F-1	1	TU4	690	2.50	690	Report dated 9/8/71 (DH-4-36P)
F-1	2	TU4	230	2.0	230	Report dated 9/8/71 (DH-4-36)
F-1	3	TU4	580	1.4	580	Report dated 9/8/71 (DH-4-36)
F-1	1	Dr. 3A	231	2.0	231	Report dated 3/10/72, Exit
F-1	2	Dr. 3A	58	3.0	58	Report dated 3/10/72, Exit
F-1	1	Dr. 3B	462	0.4	462	Report dated 3/10/72, No exit
F-1	1	TU5	58	250	58	Report dated 3/10/72
F-1	1	TU5	115	16	115	Report dated 3/10/72
F-1	1	Dr. 5A	58	52	58	Report dated 3/10/72, Exit
F-1	1	Dr. 5A	231	43	231	Report dated 3/10/72, Exit
F-1	1	Dr. 5A	231	25	231	Report dated 3/10/72, Exit
F-4	1	TU5	462	5	462	Report dated 3/10/72, No exit
F-1	4	Dr. 5A	600	6	600	Report dated 4/19/72, No exit
F-1	1	Dr. 5B	807	0.4	807	Report dated 4/19/72, No exit
F-1	1	TU5	692	33	577	Report dated 4/19/72, Exit
F-1	1	TU5	173	928	70	Report dated 4/19/72, Exit
F-3	1	Dr. 2B	600	0.2	600	Report dated 4/19/72, No exit
F-3	2	TU2	461	280	58	Report dated 4/19/72, Exit
F-4	1	Dr. 5B	600	65	100	Report dated 4/19/72, results suspect
F-4	1	Dr. 5B	600	74	80	Report dated 4/19/72 Results suspects
F-4	2	TU2	807	3	807	Report dated 4/19/72, No exit
F-4	3	TU2	692	128	350	Report dated 4/19/72, Exit
F-7	1	Dr. 5A	600	406	230	Report dated 4/19/72
F-7	1	Dr. 2A	600	198	540	Report dated 4/19/72
F-7	1	Dr. 2A	346	474	180	Report dated 4/19/72, results suspect
F-8	1	TU2	600	129	120	Report dated 4/19/72, Exit
F-8	1	TU2	519	171	60	Report dated 4/19/72, Exit
F-8	1	TU2	600	0.01	600	Report dated 4/19/72, No exit
F-9	1	Dr. 5B	807	130	115	Report dated 4/19/72, Exit
F-10	2	TU2	600	49	35	Report dated 4/19/72, Exit
F-26	1	TU2	600	0.1	600	Report dated 4/19/72, No exit
F-26	1	TU2	600	0.3	600	Report dated 4/19/72, No exit

F-Zones - The F-zones in the area are individually limited in length from a few hundred to a few thousand feet. See the Site Geology Map. They have developed parallel to two major joint sets delineated by the USBR in the area. F-1 and faults subparallel to it (F-8, -10, -12, -13, -16, -17, -26, and -33) are associated with a joint set striking about N40°W and dipping 45°SW. The low angle faults (F-3, -20, -21, and -22) are associated with a joint set which strikes about N60°W and dips 15 to 20°SW.

F-1, the major fault in the foundation of the thin-arch dam, extends sinuously from the top of that dam's left abutment into the channel, passes out of the foundation and extends upstream of the right abutment. As mapped, the 4,500-foot-long surface trace of the fault lies approximately 200 feet from the left abutment of the proposed COE alignment (crest elev. 924). The strike ranges from N20°W through E-W to N80°E, and the dip ranges from 25° to 70°SW. The average strike is N65°W, and dip is 55°SW. Although the fault dips toward the southwest in the direction of the proposed COE alignment, contour maps of the top of F-1 developed by the USBR indicate that the fault lies approximately 220 feet below the centerline of the COE alignment. F-1 is described as a complex system of thin gouge zones, dikes, altered amphibolite, chlorite schist C, and quartz-calcite veins. The average cumulative thickness of sheared material in the zone is about 3 feet. The USBR reported that the fault zone generally has a well developed drag zone indicative of reverse fault movement with a maximum observed lateral displacement of 120 feet measured on a near vertical metasandstone unit in the lower left abutment of the thin-arch keyway. Studies by the USBR on the relationship of F-1 and several dikes, indicate that the maximum possible net slip on F-1 in the last 140 million years appears to be about 3 feet reverse dip slip with a slight lateral component (Project Geology Report, Summary Volume, page 34).

Smaller splay faults off of F-1 (F-1a, -1b, -4, -5, -6, -7, -9, -14, -15, -19, -28, 29, and -35) have an average strike of N75°W and dip of 50°SW. Generally these splays are limited in length to a few hundred feet, and have average cumulative shear thickness of less than 0.7 feet.

F-0, which passes northeast of both the thin-arch foundation and the COE alignment, is the longest F-zone in the area, with a surface trace of less than two miles. F-0 is subparallel to F-1, striking about N70°W and dipping steeply to the southwest. The surface trace of the fault lies several hundred feet to the northwest of the left abutment of the proposed COE flood control dam. The fault generally consists of a 3to 10-foot-wide zone of disturbed rock with one or more shears having a total cumulative thickness of about 0.5 foot. The drag and offset relationships of the T-zones by F-0 suggest left lateral movement similar to F-1.

The Maidu East shear, which is the only documented Cenozoic fault near the damsite, is associated with T-26, a sheared talc-serpentine complex that parallels the regional metamorphic fabric. This T-zone trends N-S to N30°W, is generally near vertical, and ranges from less than 20 feet to more than 300 feet in width. The surface trace of the

T-zone extends from the river channel about 1 1/2 miles south of the damsite, to an area approximately one mile north of the site, and passes approximately 700 feet west of the right abutment of the proposed COE flood control dam. A trenching investigation conducted by the USBR indicates that the Maidu East shear is the only fault within the T-zone that offsets rock of the overlying Tertiary-aged Mehrten formation. The fault is reported to exhibit vertical offsets and slickensides which indicate displacement is down to the east, normal faulting with a right-lateral component.

Weathering - A considerable amount of attention was given to the determination of weathering in bedrock in the USBR explorations for their thin-arch dam. Weathering was the primary consideration for determining the depth of excavation, cut slope design, and utilization of excavated material.

The USBR determined that differential weathering occurs throughout the rock units due primarily to the structural and lithologic character of the rock. The relatively high permeability along certain joints, cleavage planes, and minor shear zones allow weathering to penetrate more deeply into parts of a single rock unit, whereas the differences in the susceptibility to the breakdown of rock due to it's compositional or textural differences causes differential weathering of unlike rock units. Table N-3-26 lists the general resistance and susceptibility to weathering of the rock units in the Auburn area:

TABLE N-3-26

RELATIVE WEATHERING RESISTANCE OF ROCK UNITS

More Resistant	Quartz
	Talc Schist
	Chlorite Schist
	Serpentine
	Metagabbro
	Amphibolite
	Metasediments
	Dikes
Less Resistant	Calcite

It is anticipated that foundation treatment for the proposed dam will include the removal of weathered rock to the depth required for the foundation to be seated on slightly weathered rock. Due to the differential weathering characteristics described above, those zones in which the bedrock will consist of metasediments and talc zones will require substantially more excavation of material to reach slightly weathered rock than those areas underlain by amphibolite.

Discontinuities - Discontinuities, or the naturally occurring breaks in the rock mass, are present throughout the area in the form of shear zones, faults, joint sets, and cleavage planes.

Shear zones, which are relatively planar zones of fragmented rock that usually contain some clay gouge, occur with varying frequency and continuity throughout the project area. The USBR found that most shears crosscut the foliation and dip to the southwest. They varied from single thin layers of clay gouge, continuous for less than 100 feet, to thick zones containing clay gouge, quartz-calcite veins, and dikes which may run for more than 1000 feet. Those shear zones within amphibolite and metasediments parallel the foliation of the rock and tend to be the least continuous, on the order of less than 100 feet. Within the T-zones, shearing parallels the foliation and tends to be on the order of several hundred feet in length. The USBR has produced two computerized contoured shear density diagrams as upper hemisphere equal-area projections. These diagrams delineate the shears in Tunnels 2 and 4 and are conventional diagrams with no correction for orientation or length of traverse. Eight diagrams with corrections were produced and they cover specific blocks within the various drifts and tunnels. The diagrams were prepared as an aid to understanding the geology of the site and are not complete with title blocks. The original data from which the diagrams were produced is not presently available from the USBR, and there are no reproducibles. Consequently, the diagrams are presented as received from the USBR. The locations from which the data was gathered is shown beneath each hemisphere. These shear diagrams are presented in Appendix N-3-A. The USBR identified twenty-six crosscutting shear zones over large portions of their study area, which, because of their cross cutting nature, are referred to as faults, or F-zones (F-0, F-1, etc.). F-zones which project into the area of the proposed COE alignment are shown on the Geologic Section Along Axis of Dam (Plate 11). (NOTE: Plate 11 is too large to include in the appendix. It will be sent in a separate package to reviewers. Others who wish to view this plate may do so at the Geology Section of the Sacramento District.)

Joint Sets - A joint is a surface of fracture or parting in a rock without displacement. A joint set is a series of joints with similar or parallel orientation. During the preconstruction foundation investigations at the proposed Auburn thin-arch damsite, a series of nine joint sets were encountered. Pertinent information on these joint sets was tabulated and is presented in Table N-3-27. The data for each joint set includes average attitude, average and range of the spacing and length of the sets, and the percent of joints in a set that are healed by quartz and/or calcite. The "percent areal distribution" was also calculated. This is the ratio of the sum of the tunnel and drift intervals over which a set was observed to the total length of tunnels and drifts in the foundation block.

TABLE N-3-27

AUBURN DAMSITE JOINT SET DATA FROM EXPLORATORY EXCAVATIONS								
JOINT SET	AVERAGE STRIKE	AVERAGE DIP	SPACING		LENGTH*		PERCENT HEALED*	PERCENT AREAL DISTRIBUTION
			AVG.	RANGE	AVG.	RANGE		
A-1	N57°W	17°SW	1.4'	0.8-3.0'	10'	2-150'	60	36%
A-2	N64°E	16°SE	1.8'	0.6-4.0'	8'	4-29'	65	19%
A-3	N60°W	15°NE	1.3'	0.7-2.0'	10'	6-27'	ND#	6%
A-4	N48°E	20°NW	1.7'	0.5-3.0'	10'	6-27'	80	19%
D-1	N59°W	44°SW	1.6'	0.5-5.0'	11'	2-50'	76	52%
D-2	N62°E	49°SE	1.8'	0.5-8.0'	9'	3-41'	65	43%
D-3	N56°W	44°NE	1.5'	0.5-2.0'	7'	5-9'	ND#	11%
D-4	N47°E	41°NW	2.0'	0.5-2.6'	8'	2-20'	65	39%
B	N66°E	86°NW	1.4'	0.6-4.0'	8'	2-35'+	40	86%

* - INCLUDES DATA FROM DIVERSION TUNNEL.
# - NO DATA AVAILABLE

Because of the wide variations in the range and average attitude of sets that occur from one location to the next, these average joint set attitudes are only statistical representatives. During construction excavation all joints encountered within a particular foundation area were tabulated and logged on special forms. A comparison of these logs with the preconstruction data shows a close correlation in the areal distribution of the joint sets.

A series of contoured joint diagrams was prepared and computerized by the USBR as upper hemisphere, equal-area projections. Nine were compiled from data gathered in tunnels and drifts and seven were made from data from specific drill holes. Although this information is site specific for the proposed axis of the USBR dam it can probably be projected to the proposed COE axis. These joint sets are under the same constraints as the shear diagrams and they are presented in Appendix N-3-B.

Cleavage planes, which are discontinuities parallel to the foliation of the metamorphic rock, have the greatest significance in the amphibolite, gneissic metagabbro, and metasediments. In the fresh and slightly weathered amphibolite and metagabbro, cleavage planes are generally spaced 0.5 to 2.5 feet and are continuous for 2 to 10 feet. Most cleavage planes are planar to slightly wavy, moderately smooth, and tight. Locally, they are slickensided or thinly clay-coated. Cleavage planes within the highly weathered amphibolite and metagabbro are spaced mostly from 1/2 inch to 0.2 foot and are more continuous due to the weathering along foliation.

Cleavage planes in metasediments are parallel to the original bedding. In general, they are more closely spaced, continuous, planar, and smoother than those in the amphibolite. Coatings of chlorite are common on cleavage planes within a few feet of T-zones. In fresh metasandstone, cleavage planes are spaced about 0.5- to 1.0 foot. In the metashale they generally are spaced 1/2- inch to 0.5 foot in slightly weathered and fresh rock, and 1/16- to 1-inch in highly weathered rock.

Groundwater - Based on explorations and excavations conducted by the USBR, groundwater at the site is anticipated to be small in volume, and occurs primarily along joints, cleavage planes, dikes, shear zones, and F-zones. The USBR encountered groundwater flows of 2 to 10 gallons per minute along F-zones and dikes and from the upslope side of T-zones. The F-zones and T-zones also act as impermeable barriers to the movement of water. Both perched and artesian conditions were encountered.

Explorations - In addition to the extensive geologic investigations and explorations for the thin-arch dam at River Mile (RM) 20.1, the USBR also conducted numerous geologic explorations as part of the feasibility studies for two alternatives to the thin-arch dam, which were to be located in the area of the COE proposed dam. See Appendix M, Chapter 5, Geologic Evaluation of Alternative Damsites, Table 1, for an outline of the current status of the geologic investigations conducted

by the USBR. Plate 12 shows explorations conducted by the USBR in the vicinity of the proposed COE dam and also the locations of cross-sections drawn by USBR and COE. COE cross-sections are shown on Plates 13 through 17. USBR cross-sections are shown on Plates 18 through 30. (NOTE: Plates 12 and 18-30 are too large to include in this appendix. They will be sent in a separate package to reviewers. Others who wish to view these sections may do so at the Geology Section of the Sacramento District.) To date, no specific explorations have been conducted for the proposed COE dam. Explorations for the proposed dam will be one of the first activities of PED. It should be noted that the level of detail obtained by the USBR is the result of a massive geologic exploration program extending over a period of six years prior to start of construction, five years during construction and two years after construction was halted. Fortunately, the northwesterly regional trend of the geologic structure such as faults, talc-zones, and rock fabric are such they can reasonably be projected from the area of study by the USBR into the area of the proposed COE dam. Additional geologic investigations including in-situ rock testing and rock core drilling will be required to better delineate the foundation conditions at the proposed damsite. Until work on the foundation excavation is actually begun, it won't be possible to match the level of detail obtained by the USBR.

Hydraulic Design

The design references used for the hydraulic design of the selected plan are the same as described for the alternatives. Rating curves were developed in the same manner as developed for the alternatives. The following information gives details on the spillway and outlet works for the selected plan. The selected plan will have a flood control storage capacity of 545,000 acre-feet at a spillway crest elevation of 868.5 feet.

Since the present design is for flood control only, with no permanent pool, sedimentation effects will be significantly different from that which would occur under permanent pool conditions. Significantly more of the finer sediments will be passed through the outlet works sluices during floods. Inflowing bedload and other coarser sediments will be transported and deposited further downstream within the reservoir area than would occur with a multiple-purpose reservoir. The amount and areal distribution of sediments over time is of great interest to designers and environmental groups. Therefore, geomorphic and sediment studies will be performed during the next study phase to address these concerns as well as possible effects downstream. Prior sediment yield studies by the USBR have shown that the average annual sediment yield is relatively small. The estimated 100-year sediment accumulation in their proposed multiple-purpose reservoir was 26,200 acre-feet. Additional discussion on sediment inflow is given in the Hydrology Appendix, Chapter VI, Section 7. All sluice intakes are currently proposed to be located above the minimum sill elevation of

482 feet suggested by the USBR. This elevation accounted for 100 years of sediment deposition in a permanent pool, making the simplified assumption of a flat sediment pool.

Spillway. - The spillway crest elevation is set such that only flows in excess of the design level of protection (200-year) will pass over the spillway. The spillway crest was designed as an ungated standard ogee with an elliptical upstream quadrant and a parabolic downstream quadrant. The downstream quadrant becomes tangent to the spillway chute at a slope of 1.0V on 0.7H. A flip bucket will be used to insure that energy dissipation occurs a safe distance downstream of the toe of dam. The flip bucket invert is located above the PMF tailwater in order to insure proper functioning. Future studies, including a physical model study will determine if this is the optimum elevation for the flip bucket lip. Plate 9 shows a profile of the spillway. A 600 feet spillway crest length was selected. This size lowered the total height of dam while still allowing the jet from the spillway flip bucket to fit in the canyon downstream. The spillway rating curve was computed by assuming a high ogee weir condition and using the following equation:

$$Q = C\ [L' - 2(NKp+Ka)He]\ He^{3/2}$$

Where:

Q = Discharge (cfs)	Kp = Pier Contraction Coefficient
C = Discharge Coefficient	Ka = Abut Contraction Coefficient
L = Net Length of Crest (ft)	He = Total Head on Crest (ft)
N = Number of Piers	

Discharge coefficients were taken from HDC Sheet 111-3. Abutment contraction coefficients are presented in HDC Sheet 111-3/2. Although Sheet 111-3/2 is for embankment abutments, the values obtained are more conservative than those for concrete abutments. Also, the embankment coefficients rely on the better defined ratio He/Hd as opposed to He/R for concrete, R being the radius of the abutment. An additional 10 percent reduction in discharge was assumed due to the skewed approach angle to the spillway. Future model tests will determine if this assumption is reasonable. The value of N in the above equation is zero as there are no bridge piers.

The Probable Maximum Flood (PMF) was selected as the SDF according to the definition of the functional design Standard 1 presented in EC 1110-2-27. The PMF hydrograph was routed through the reservoir assuming no outlet works release and an initial pool at spillway crest elevation 868.5 feet. The routing resulted in a maximum pool elevation of 923.7 feet and a maximum spillway discharge of 860,000 cfs.

Energy dissipation for the spillway flow will be facilitated by using a flip bucket. A flip bucket was chosen because of the quality of rock downstream and the impracticality of using a roller bucket

configuration due to tailwater conditions. Tailwater elevations were modeled using the computer program HEC-2, "Water Surface Profiles." Cross sections were taken from immediately upstream of Folsom Reservoir upstream to the damsite. Topography was taken as current conditions except for immediately downstream of the Dam where pre-1986 (i.e., USBR project) elevations were assumed. Starting water surface elevations were obtained by the slope-area method.

Due to the curved axis of the dam, a converging spillway is necessary. The spillway will converge as it drops from the crest to the flip bucket. The converging spillway walls will be straight and the rate of convergence will conform to EM 1110-2-1603. The spillway will converge from 600 feet at the crest to approximately 530 feet at the flip bucket.

Outlet Works. - The original concept used for analyzing the alternatives consisted of using the existing diversion tunnel and sluices through the dam as the outlet works. The existing diversion tunnel (after modification) would be used to discharge as much of the flood flow as possible, while the sluices discharged the remaining flow. After further consideration, it has been determined that the addition of emergency gates to the outlet works would enhance system safety. In the unforeseen event that the downstream levees developed distress, the emergency gates could be used to reduce flood flows from the dam and provide some additional relief downstream. For these reasons, the selected plan will include emergency gates in the outlet works.

The provision of emergency gates for the diversion tunnel would be very costly. A large control tower and very large gates would be required. Therefore, the outlet works concept for the selected plan was revised to consist of multiple flood control sluices through the dam. Each sluice would be provided with an emergency gate and provisions for bulkheading at the intake. Six sluices would be placed on each side of the spillway section of the dam. The emergency gates would be operated from a gallery within the dam. The selected plan will include 12 - 5.0 feet by 9.5 feet sluices to serve as the flood control outlet works. The diversion tunnel would be bulkheaded and would not be used for flood control discharges. Bulkheading of the tunnel is less expensive than plugging. Five pairs of the sluices, a total of ten of the sluices, will transition into larger sluices, 17.0 feet wide by 12.5 feet high, downstream of the emergency gates, see Plate 31. The remaining pair of sluices will continue through the dam as separate sluices. This transition results in twelve sluices upstream of the gates and seven sluices downstream of the gates. The pair of sluices which do not transition will be set at the same elevation as the existing diversion tunnel, elevation 491, to maintain existing low flow characteristics and limit any erosion/abrasion damage, caused by bedload materials, to the other sluices. The other sluices will be set at elevation 500. Each sluice will have a single emergency slide gate capable of being operated under high heads. Operation will take place from a gate chamber within the dam as shown on Plates 9 and 31. Air systems will be provided for

each gate. A backup closure system will consist of bulkhead slots for each sluice with a bulkhead and frame ensemble stored on the side of the dam at both ends of the spillway, for a total of two. Should trouble develop with an emergency gate, a portable crane will swing the bulkhead and frame into the slots and lower the ensemble down to the opening of the sluice. The slots are designed with dogs above the sluice entrances. When the bulkhead frames reach these dogs, automatic locking devices will extend into the dogs and secure the bulkhead frame in place. The frame will be equipped with an underwater electric motor driving a hydraulic or gear reduction system capable of forcing the bulkhead closed under high flows.

The emergency gates will remain open at all times until an emergency occurs. During flood events, both the Sacramento District and the California Department of Water Resources (DWR) go into flood operation status and closely monitor the flood control system. The operating manual for the dam will describe what constitutes an emergency and how a decision to close the emergency gates will be made. The preliminary concept is to require a joint decision of the Corps and DWR flood system operators, in coordination with the Bureau of Reclamation, to make a decision to close the emergency gates at the flood control dam. Gates would remain closed until the emergency situation was under control or eliminated. A more detailed description of an emergency scenario and operation is given in the Special Topics Chapter of the Main Report.

The sluices have trash struts, intakes suitable for high head discharges, and flip buckets at the outlets to force energy dissipation well downstream from the toe of the dam. The flood control sluice intakes will have elliptical intake curves for both roof and walls. Plate 32 shows details of the sluice intake. The outlet works discharge capacity was calculated to control the design flood inflow hydrograph to an acceptable level for Folsom Dam and Reservoir and the lower American River system. Drawdown criterion is not a controlling factor for outlet works releases as the flood pool will reach ten percent of its ultimate storage volume in far less than the 120 days allowed by ER 1110-2-50. The technical design criteria presented in EM 1110-2-1602, "Hydraulic Design of Reservoir Outlet Works", and HDC 200 - Outlet Works, were used for sizing and rating of the outlet works. The sluices will transition to flip buckets at their exit so that energy dissipation will occur downstream of the outlet works with the tailwater acting adding depth to the plunge pool. Plate 33 shows details of the flip bucket geometry.

The energy of flows from the sluices and the spillway will dissipate in a plunge pool downstream. This area is underlain by amphibolite and is considered competent to withstand the design flows. The Site Geology section discusses this rock unit in more detail. No problems are expected from the range of flows which should be experienced at the site.

Structural Analysis

Purpose and Scope. - Seismic analysis was performed as part of the structural analysis to provide feasibility-level design for a roller compacted concrete (RCC) gravity dam at the RM 20.1 site. Specifically, an earthquake response analysis of the selected plan (200 year flood control dam) is presented in this section.

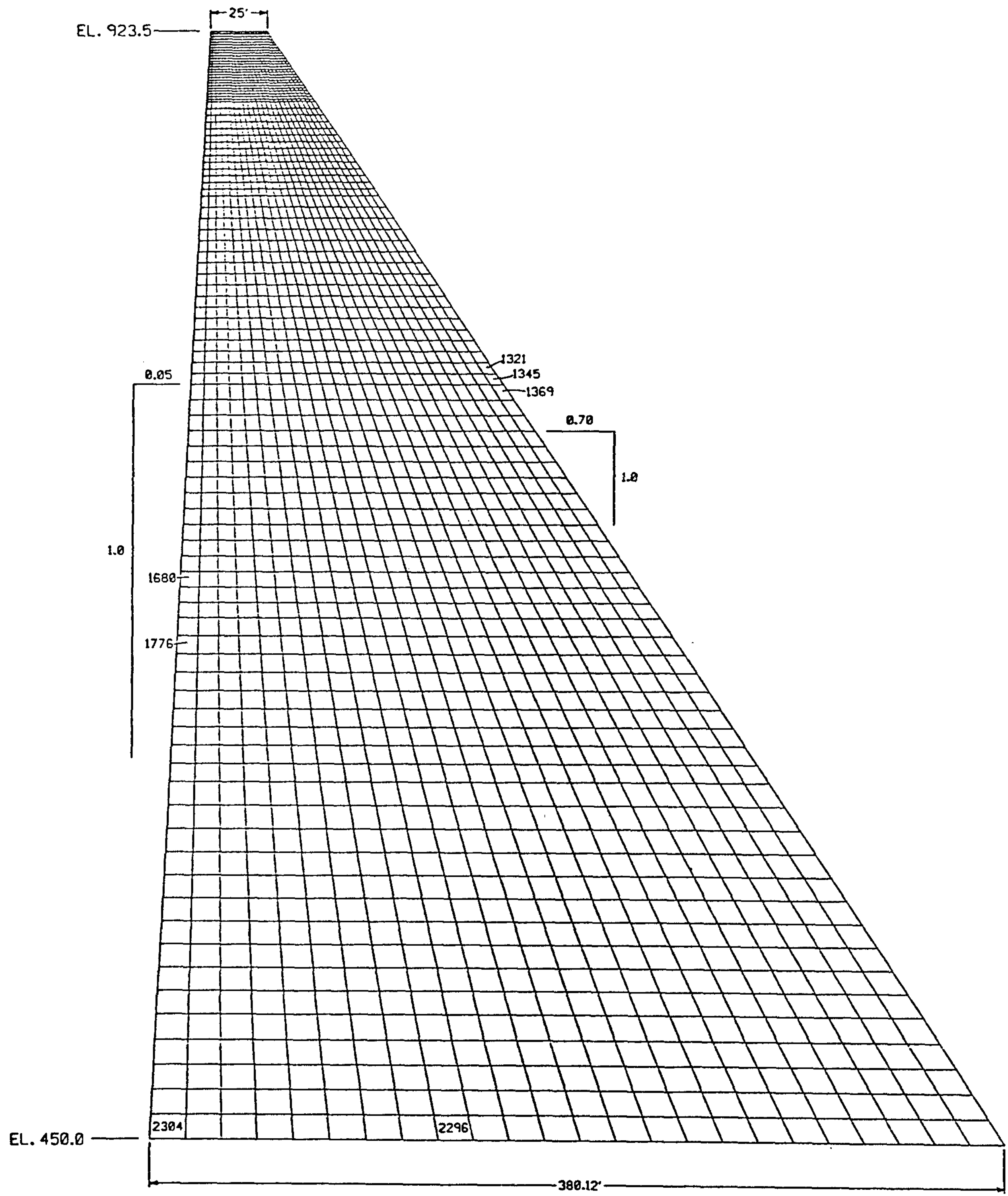
Computer Program. - Two-dimensional finite element analyses of the proposed dam were performed with the use of the computer program EAGD-84. This program is designed specifically for two-dimensional earthquake analysis of concrete gravity dams [7,8,9]. The program was developed at the University of California at Berkeley by Dr. Gregory Fenves and Professor Anil K. Chopra. A detailed description of input data for the program is available in reference [10].

Basically, EAGD-84 performs a substructure analysis in the frequency domain and computes the response of a concrete gravity dam subjected to an arbitrary earthquake ground motion. The simultaneous effects of dam-reservoir interaction, dam-foundation rock interaction and reservoir bottom wave absorption are included in the analysis.

Dam Finite Element Model. - During large-amplitude earthquake vibration, the inertia forces developed are much greater than the shear forces that can be transmitted across joints between the dam monoliths. Consequently, the joints would slip and the monoliths tend to vibrate independently. Considering the effects of the joints, a two-dimensional plane stress model of the individual monoliths is appropriate for predicting the earthquake response. The tallest, non-overflow monolith of the flood control dam was selected for the finite element analysis.

The dam monolith is idealized as an assemblage of planar, four-node non-conforming finite elements connected at the nodal points in the global X-Y plane. The X-axis is horizontal and positively directed downstream; the Y-axis is vertical with the positive direction upwards. The nodes at the base of the dam must be equally spaced in order to consider the effects of dam-foundation rock interaction. The finite element mesh should be laid out in such a way that element aspect ratios should not be excessive. They should be on the order of 1:1, and preferably less than 4:1. Elements in the shape of parallelograms with aspect ratio near unity usually give the most accurate analysis results. The smallest bandwidth of the structural stiffness matrix, and hence lower computational cost can be achieved by numbering the nodal points in the direction of the monolith cross-section with the smallest number of elements. However, the numbering of elements can be arbitrary because it has no effect on the computational effort.

The finite element model for the flood control dam is shown in Figure 18. This dam mesh was determined to be sufficiently refined to give accurate stress results. Note that two translational degrees of freedom (DOF) are associated with each free node.



FINITE ELEMENT MESH OF THE PROPOSED AUBURN RCC GRAVITY DAM
200 YEAR FLOOD PROTECTION DAM

FIGURE 18

Considering foundation-rock flexibility, the nodes at the base of the dam are free to translate in the X, Y-directions. The features of the finite element idealization are summarized as follows:

Number of Nodes At Dam Base	Total Number of Nodes	Total Number of Elements	Total Number of DOF
25	2425	2304	4850

Foundation-rock Model. - The foundation rock supporting the dam is idealized as a homogeneous, isotropic, viscoelastic half-plane [8,9,11,12]. This half-plane idealization permits accurate modeling of damsites where uniform foundation materials extend to large depths under the dam. In order to define the dam-foundation system on a consistent basis, a plane stress model is employed for the foundation rock. This model, though not strictly appropriate for the foundation, is dictated by the expected behavior of the joints between the monoliths, as discussed under "Dam Finite Element Model" above. For the purpose of including dam-foundation interaction effects, the foundation surface is assumed to be horizontal. The damsite in this study is idealized to conform to this assumption.

Hydrodynamic Effects. - Hydrodynamic pressures in excess of the usual hydrostatic pressures are developed in the reservoir due to the earthquake ground motions and deformations of the upstream face of the dam. The structural deformations are in turn affected by the hydrodynamic pressures acting on the dam. The hydrodynamic effects need to be properly modeled to recognize the dynamic interaction between the dam and water.

The analysis procedure implemented in EAGD-84 efficiently evaluates the dam-reservoir interaction during an earthquake. In this procedure, the water impounded in the reservoir is idealized as a continuum of constant depth and infinite length in the upstream direction [7,8,9]. The dynamic response of the dam with impounded water is affected to a significant degree by the hydrodynamic terms in the equations of motion for the dam. These terms are determined from the solution of the wave equation for appropriate accelerations at the boundaries of the fluid domain. The hydrodynamic terms can be interpreted as modifying the properties of the dam by introducing an added mass, an added force, and an added damping. The following properties for the impounded water are used: velocity of pressure waves in water = 4720 ft/sec, and unit weight = 62.4 lb/ft3.

Material Properties

Dam Concrete. - The U.S. Bureau of Reclamation (USBR) conducted a concrete testing program for their authorized Auburn Dam in the 1970s. However, no laboratory and field tests on RCC were performed. Based on a literature search into previous testing on RCC, values of concrete properties were compiled [14]. In this study, the mass concrete in the dam is assumed to be homogeneous, isotropic, and linearly elastic with the following properties from reference [14] used for the dynamic analysis, Table N-3-28.

TABLE N-3-28

PROPERTIES OF DAM CONCRETE FOR SELECTED PLAN
FOR
EARTHQUAKE FINITE ELEMENT ANALYSIS

Static f'c (psi)	Poisson's Ratio Vc	Unit Weight Yc (kip/ft^3)	Young's Modulus of Elasticity Ec (ksi) Lower Bound	Young's Modulus of Elasticity Ec (ksi) Upper Bound
3000	0.2	0.150	2000	3900
5000	0.2	0.150	(Not used)	4900

In the absence of laboratory data for the Young's modulus of concrete Ec, Ec is varied over a range (2000, 3900, 4900 ksi) to evaluate the effects of dam stiffness on the stress response of the dam.

To assess the seismic performance of the proposed dam, the strength properties shown in Table N-3-29 were used [14]. In judging the safety of the dam, it is appropriate to compare the computed tensile stresses with the "apparent tensile strength" of the concrete [15]. The apparent tensile strength is equal to the splitting tensile strength augmented by a factor of approximately 1.3 to take into account the nonlinearity of concrete.

TABLE N-3-29

STRENGTH PROPERTIES OF DAM CONCRETE
FOR SELECTED PLAN

Static f'c (psi)	Dynamic Compressive Strength (psi)	Apparent Dynamic Tensile Strength (psi)
3000	3900	580
5000	6500	990

<u>Foundation Rock</u>. - No recommended value for the deformation modulus (or Young's modulus) of the foundation rock was provided in the geology reconnaissance report [1] prepared by the Corps of Engineers. A Young's modulus of elasticity Ef = 2500 ksi was used for the finite element analysis in USBR's report [6]. In this study the Young's modulus is varied: Ef = 2500 and 3750 ksi. Based on available data for the foundation rock at the damsite, Ef is believed to be in the 2500 - 3750 ksi range. The use of Ef = 3750 ksi also provides a basis for evaluating the effects of foundation stiffness on the stresses in the dam.

It was determined that small variation in the unit weight of the foundation rock has little effect on the dynamic response of the dam. Thus, a typical value for the unit weight gf = 0.165 kip/ft3 was assumed in this investigation.

Parameters for Earthquake Response Analysis

<u>Constant Hysteretic Damping Factor</u>. - Energy dissipation in the dam concrete is represented by constant hysteretic damping with a damping factor hc. A viscous damping ratio c is taken to be the same for all the natural vibration modes of the dam supported on rigid foundation rock with no impounded water. This damping ratio c corresponds to a constant hysteretic damping factor given by hc = 2c [8]. A constant hysteretic damping factor of hc = 0.14, which corresponds to a 7 percent viscous damping ratio for all natural vibration modes of the dam on rigid foundation rock with empty reservoir, was selected. hc = 0.14 is a reasonable value for the strong ground motion considered in this study and the relatively high stresses expected in the dam during the earthquake.

The constant hysteretic damping factor hf for the foundation rock can be determined from experimental tests of appropriate rock samples subject to harmonically varying stress and strain. Since no experimental hf value is available, hf = 0.25 is assumed to represent

the damping properties of the foundation rock. This damping factor is appropriate for this stage of study.

Wave Reflection Coefficient - The bottom of a reservoir typically consists of layers of alluvium, silt, and other sedimentary materials. Hydrodynamic pressure waves impinging on such materials will partially reflect back into the water, and will partially be absorbed into the underlying layers of reservoir-bottom materials. In general, the dynamic response of the dam decreases with increasing wave absorption at the reservoir bottom.

The absorptiveness of the reservoir-bottom materials is characterized by the wave reflection coefficient a, which is defined as the ratio of the amplitude of the reflected hydrodynamic pressure wave to the amplitude of a normally propagating pressure wave incident on the reservoir bottom [9]. A rigid reservoir bottom (a = 1.0) indicates that pressures waves are reflected from the reservoir bottom without attenuation; whereas a completely absorptive reservoir bottom (a = 0) means that normally propagating pressure waves are fully absorbed into the reservoir bottom materials without reflection. The wave reflection coefficient a can be determined according to the following equation [9]:

$$a = \frac{1 - qC}{1 + qC}$$

in which C is the velocity of pressure waves in water, q is the damping coefficient of the reservoir-bottom materials and is given by

$$q = \frac{r}{Pr(Er/Pr)^{1/2}}$$

where r is the mass density of water, Er is the Young's modulus of elasticity and Pr is the mass density of the materials at the reservoir bottom.

Taking Er = Ef = 2500 ksi for the foundation rock (see discussion above), the above equations lead to an (a) value of 0.65. Based on the recommendation from reference [13], a = 0.90 was used in the dynamic analysis of the proposed flood control dam, as it provides conservative estimates of stresses in the dam.

Number of Vibration Modes - To produce accurate earthquake response of the dam, all vibration modes that significantly contribute to the dynamic response should be included. Verification that enough vibration modes are included is by ascertaining that the stresses in the dam do not change if the number of modes is increased. In this investigation, 15 vibration modes were determined to be sufficient for accurate analysis of the dam. Computed natural frequencies for the first 10

modes of vibration of the dam-foundation rock system with empty reservoir are shown in Table N-3-30. A comparison of the natural frequencies in this table leads to the following conclusions:

1. Keeping the stiffness of the dam concrete fixed, the natural frequencies of individual vibration modes generally decrease as Ef decreases, i.e. the periods of vibration lengthen as the foundation rock becomes increasingly flexible.

2. Keeping the stiffness of the foundation rock fixed, the natural frequencies of individual vibration modes generally decrease as Ec decreases, i.e. the periods of vibration lengthen as the dam becomes increasingly flexible.

EAGD-84 is unable to directly compute the resonant frequencies of the dam including dam-water interaction. These frequencies, which are known to be affected by dam-water interaction, have been investigated in depth in Dr. Fenves research study [9] and will not be re-investigated here.

TABLE N-3-30

NATURAL FREQUENCIES OF DAM-FOUNDATION SYSTEM WITH NO IMPOUNDED WATER FOR SELECTED PLAN

Natural Frequencies in Hz

Mode No.	Ef = 2500 ksi			Ef = 3750 ksi		
	Ec = 2000 ksi	Ec = 3900 ksi	Ec = 4900 ksi	Ec = 2000 ksi	Ec = 3900 ksi	Ec = 4900 ksi
1	1.60	1.88	1.96	1.73	2.10	2.21
2	3.46	3.81	3.90	3.90	4.42	4.56
3	4.07	4.99	5.28	4.32	5.43	5.82
4	6.80	8.41	9.08	7.28	9.05	9.73
5	9.41	12.44	13.73	9.80	12.84	14.13
6	10.37	13.69	15.13	10.83	14.13	15.54
7	12.46	16.74	18.55	12.84	17.10	18.93
8	13.64	17.61	19.18	14.14	18.52	20.19
9	15.11	20.21	22.39	15.49	20.72	22.89
10	16.21	21.96	24.43	16.63	22.33	24.78

Fourier Transform Parameters - The earthquake response of the dam is obtained by Fourier synthesis of the complex-valued frequency response functions using the Fast Fourier Transform (FFT) algorithm. To ensure that EAGD-84 computes accurate dynamic response, the Fourier Transform parameters must be properly selected to satisfy the criteria summarized later in this section. The parameters used in the FFT computations are as follows:

Number of excitation frequencies N = 2048

Time interval for response history computation Dt = 0.01 sec.

Duration of response history T = N x Dt = 20.48 sec.

Maximum excitation frequency represented F = N / 2T = 50.0 Hz.

The following criteria [10] govern the selection of the above parameters:

- o Df $\leq$ f1 / 50.................... (1)
- o T > 1.5 / (hc x f1)................(2)
- o F $\leq$ 2.5Cf / (p x s)................(3)
- o F > fh............................(4)

where

Df is the frequency increment given by Df = 1 / 2T

f1 is the fundamental frequency of the dam-foundation rock system in Hertz

hc is the constant hysteretic damping factor for the dam concrete

s is the distance between the equally spaced nodal points at the dam base

fh is the frequency of the highest vibration mode included in the analysis

Cf is the shear wave velocity given by $(Gf/Rf)^{1/2}$, Gf is the elastic shear modulus and

Rf is the mass density of the foundation rock

Seismic Design Event - The seismic design event used for the analysis of the selected plan is the one developed at the end of a tremendous amount of study by the USBR. The adopted event was the result of several eminent board's analysis and was adopted by both the Department of Interior and the California State Geologist. A chronology of the seismic studies and analyses accomplished is given in Chapter 5 of the Geotechnical Appendix. The adopted parameters for the Maximum Credible Earthquake are as follows:

1. A magnitude 6.5 earthquake with a response acceleration of 0.5g in the one second portion of the spectrum.

2. A fault slip in the foundation of up to 9 inches.

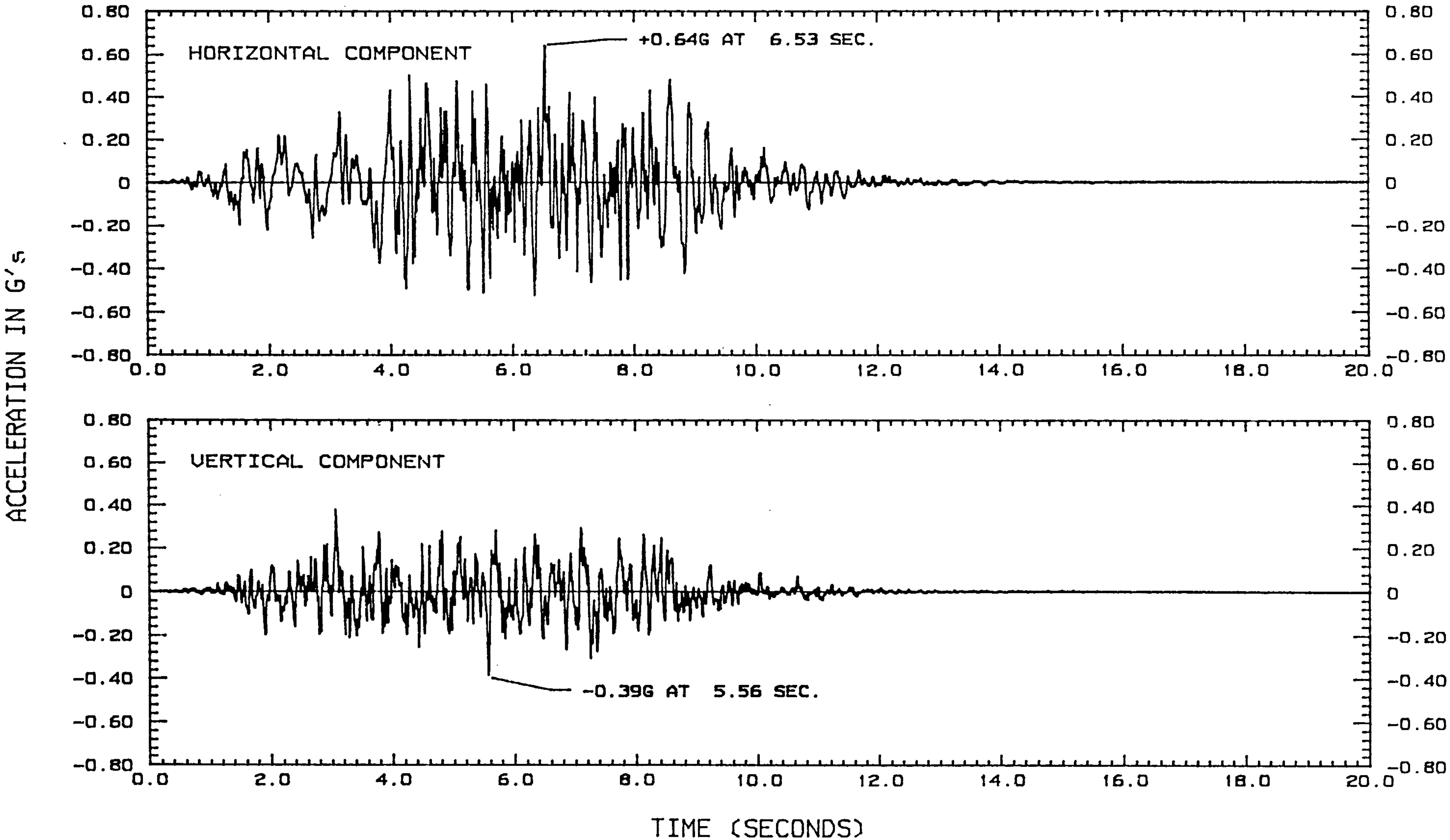
These parameters are considered conservative for this site. During detailed design, the previous studies will be reviewed with respect to any new methods of analysis or data to determine if the seismic design parameters should be modified.

Earthquake Excitation - The Bureau of Reclamation developed synthetic earthquake accelerograms used to model the Maximum Credible Earthquake (MCE) for the RM 20.1 damsite [6]. Basically, the accelerograms were generated to match the selected site response spectra. The time histories of ground acceleration in the horizontal (upstream-downstream) and vertical directions are shown in Figure 19. The response spectra for the horizontal and vertical components are displayed in Figures 20 and 21, respectively.

The earthquake excitation for the dam-water-foundation rock system is defined by the above two components of ground acceleration at the base of the dam: the horizontal component transverse to the dam axis, and the vertical component. This earthquake, having a peak acceleration of 0.64g in the horizontal direction and 0.39g in the vertical direction, provides an upper bound to feasible ground motions for the seismic analysis.

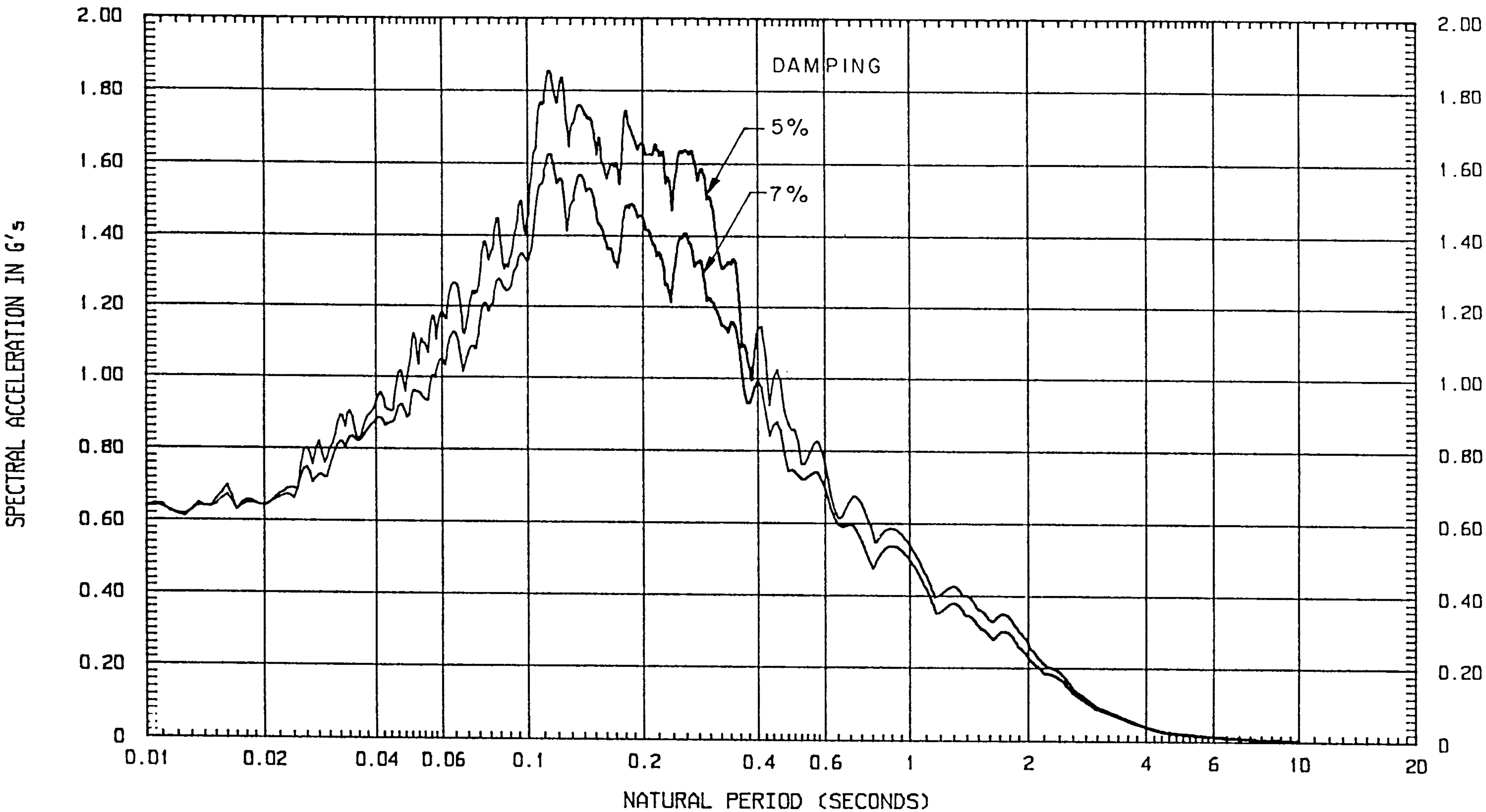
Dynamic Response Results and Evaluation - The earthquake response of the selected plan dam was computed for the simultaneous excitation of the horizontal and vertical ground motions, under the following flood pool condition - spillway crest elevation 868.5.

Because of the immense volume of response results generated from the time-history earthquake analyses for different combinations of Ec and Ef, only a small portion of the results is presented here to highlight the more important information. Stresses reported are total stresses computed at the centroid of the finite elements, including the initial static stresses due to dead weight of the dam and hydrostatic pressure.



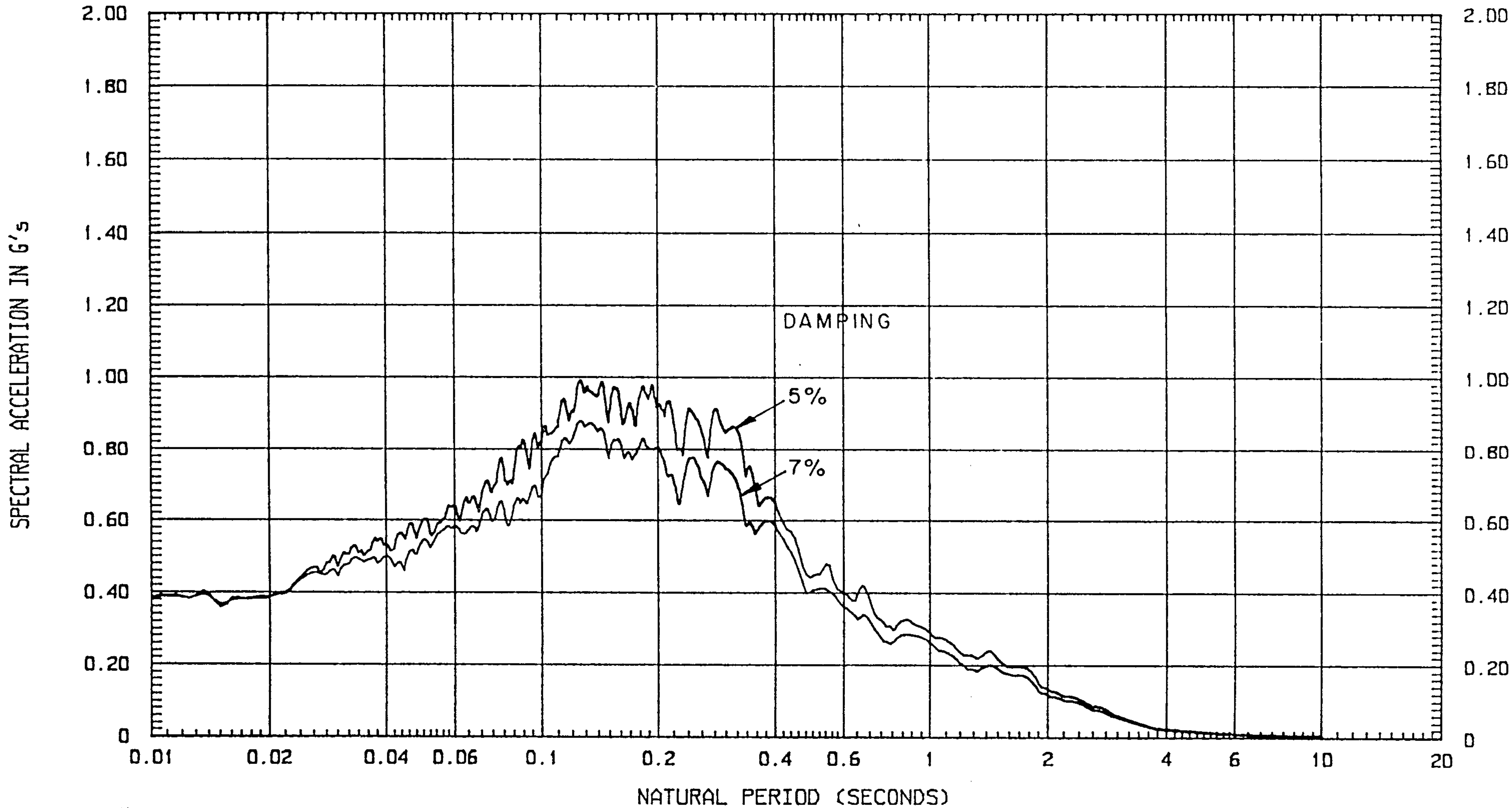
HORIZONTAL AND VERTICAL COMPONENTS OF THE MAXIMUM CREDIBLE EARTHQUAKE FOR SEISMIC ANALYSIS OF THE PROPOSED AUBURN DAM.

FIGURE 19



ACCELERATION RESPONSE SPECTRA FOR THE HORIZONTAL COMPONENT OF THE MAXIMUM CREDIBLE EARTHQUAKE. DAMPING RATIOS = 5 AND 7 PERCENT.

FIGURE 20



ACCELERATION RESPONSE SPECTRA FOR THE VERTICAL COMPONENT OF THE MAXIMUM CREDIBLE EARTHQUAKE. DAMPING RATIOS = 5 AND 7 PERCENT.

Table N-3-31 summarizes the largest compressive stresses along with the corresponding stress locations and times of occurrence. Note that element 2304 in Table N-3-31 corresponds to the upstream heel area of the dam (see Figure 18 for element numbers). These maximum compressive stresses are well within the assumed dynamic compressive strength, thus providing a large margin of safety against compressive failure.

TABLE N-3-31

SUMMARY OF MAXIMUM COMPRESSIVE STRESSES
FOR THE SELECTED PLAN

Young's Modulus of Dam Concrete Ec (ksi)	Finite Element	Time of Occurrence (Second)	Compressive Stress (psi)
Young's Modulus of Foundation Rock Ef = 2500 ksi			
2000	2304	4.85	1086
3900	2304	6.74	1330
4900	2304	6.72	1450
Young's Modulus of Foundation Rock Ef = 3750 ksi			
2000	2304	4.78	1051
3900	2304	6.71	1332
4900	2304	6.70	1448

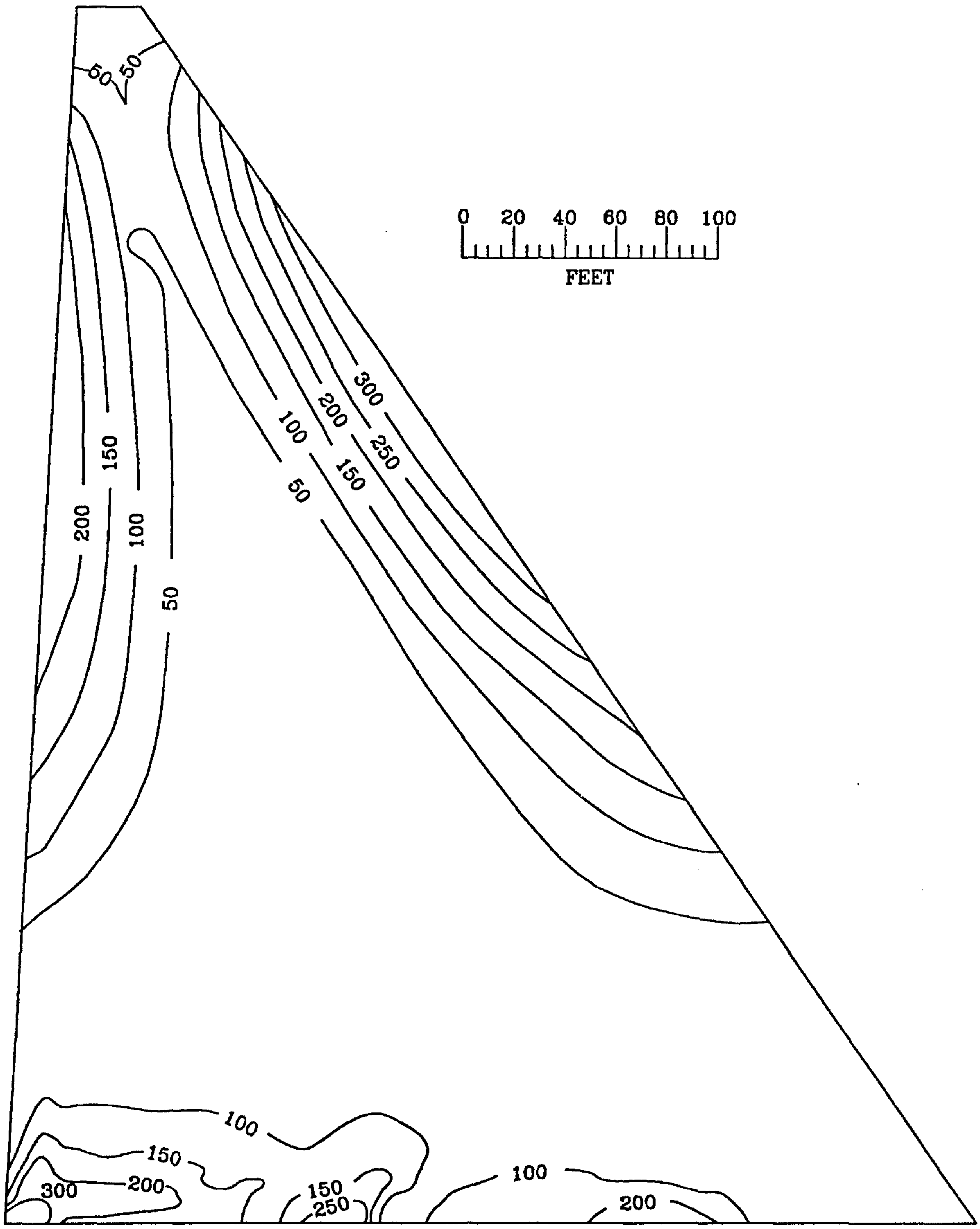
The first two maximum principal (tensile) stresses along with the corresponding stress locations and times of occurrence are compiled in Table N-3-32. It should be noted that finite elements having high tensile stresses correspond to the heel area and the faces of the dam near its mid-height.

TABLE N-3-32

SUMMARY OF MAXIMUM TENSILE STRESSES
FOR THE SELECTED PLAN

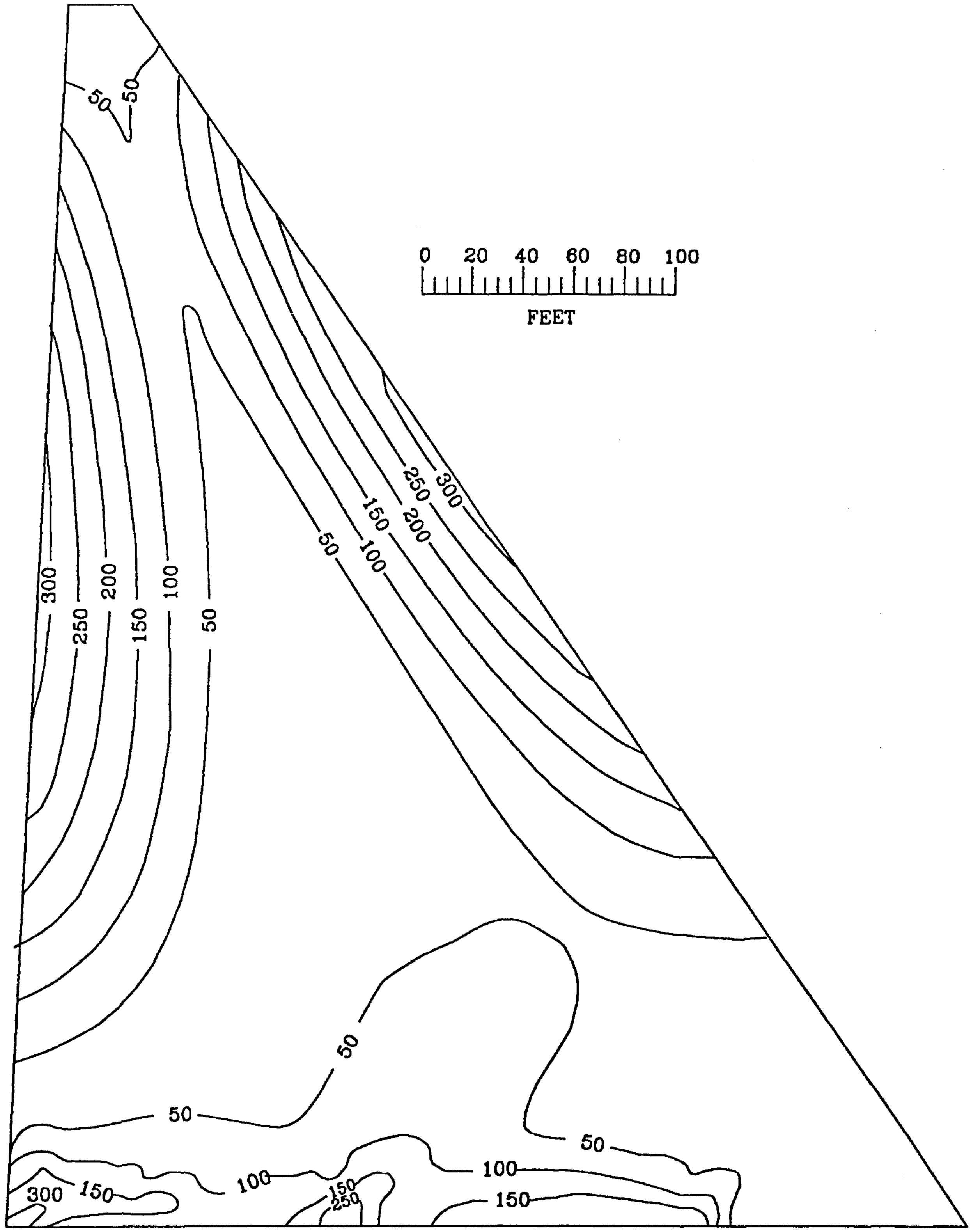
Young's Modulus of Dam Concrete Ec (ksi)	Finite Element	Time of Occurrence (Second)	Tensile Stress (psi)
Young's Modulus of Foundation Rock Ef = 2500 ksi			
2000	1369	4.18	361
	1345	4.18	360
3900	2304	4.44	341
	2296	4.32	317
4900	2296	4.32	372
	2304	4.41	327
Young's Modulus of Foundation Rock Ef = 3750 ksi			
2000	2304	4.48	441
	1321	6.74	395
3900	2304	6.93	523
	1680	4.37	386
4900	2304	6.93	581
	1776	4.36	381

Stress contours are used to display the distribution of the maximum principal stresses in the dam. Contour plots of the envelope values of those stresses are shown in Figures 22 through 27 ("envelope value" refers to the maximum value over time). Presented in Figures 28 through 33 are time histories of the maximum principal stresses at the most-stressed locations, and in Figures 34 through 39 are time histories of displacement response at the dam crest.



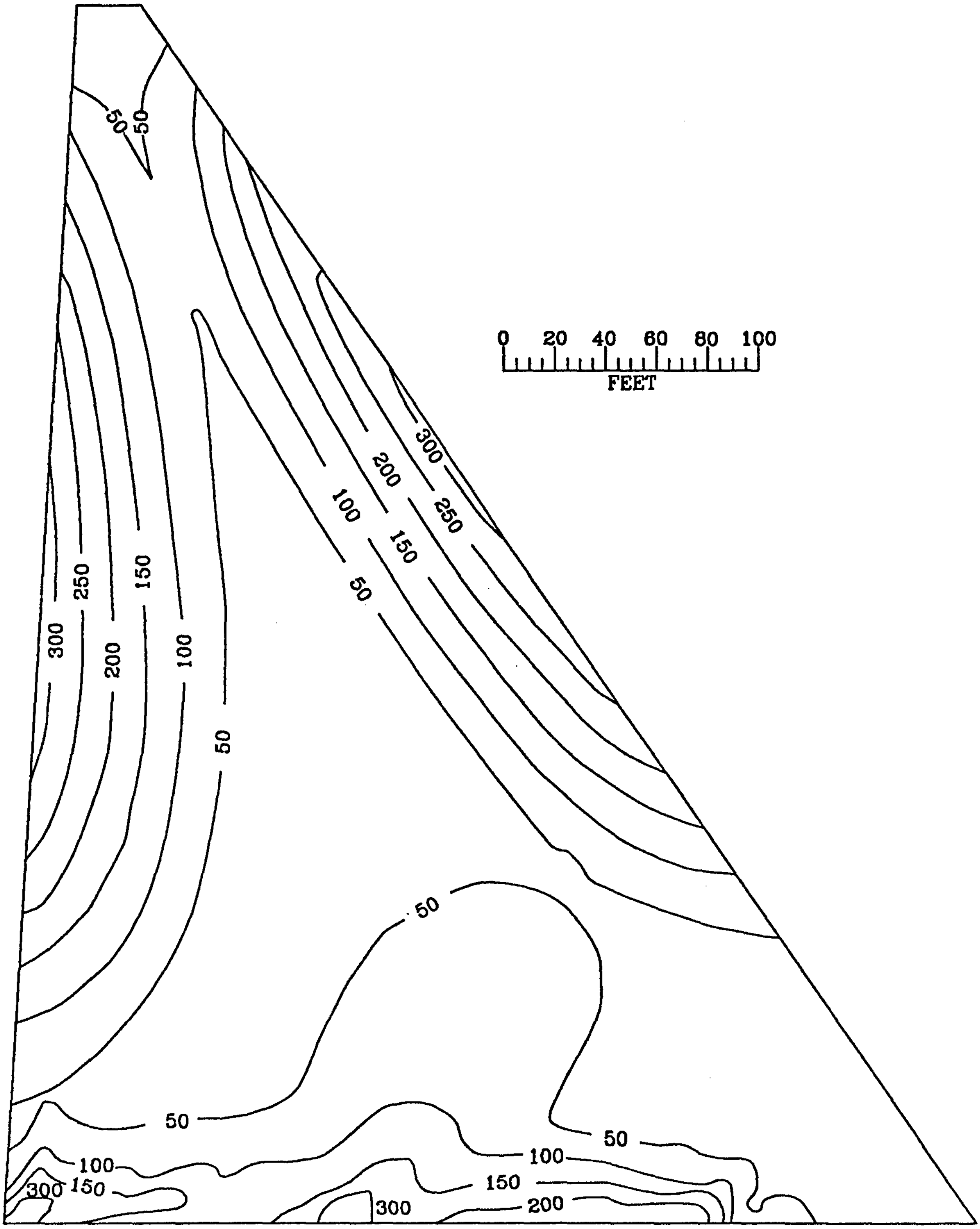
ENVELOPE VALUES OF MAXIMUM PRINCIPAL STRESSES (IN PSI) IN THE PROPOSED FLOOD CONTROL AUBURN DAM WITH POOL ELEVATION = 868.5 FEET. INITIAL STATIC STRESSES ARE INCLUDED. E_f = 2500 KSI, E_c = 2000 KSI

FIGURE 22



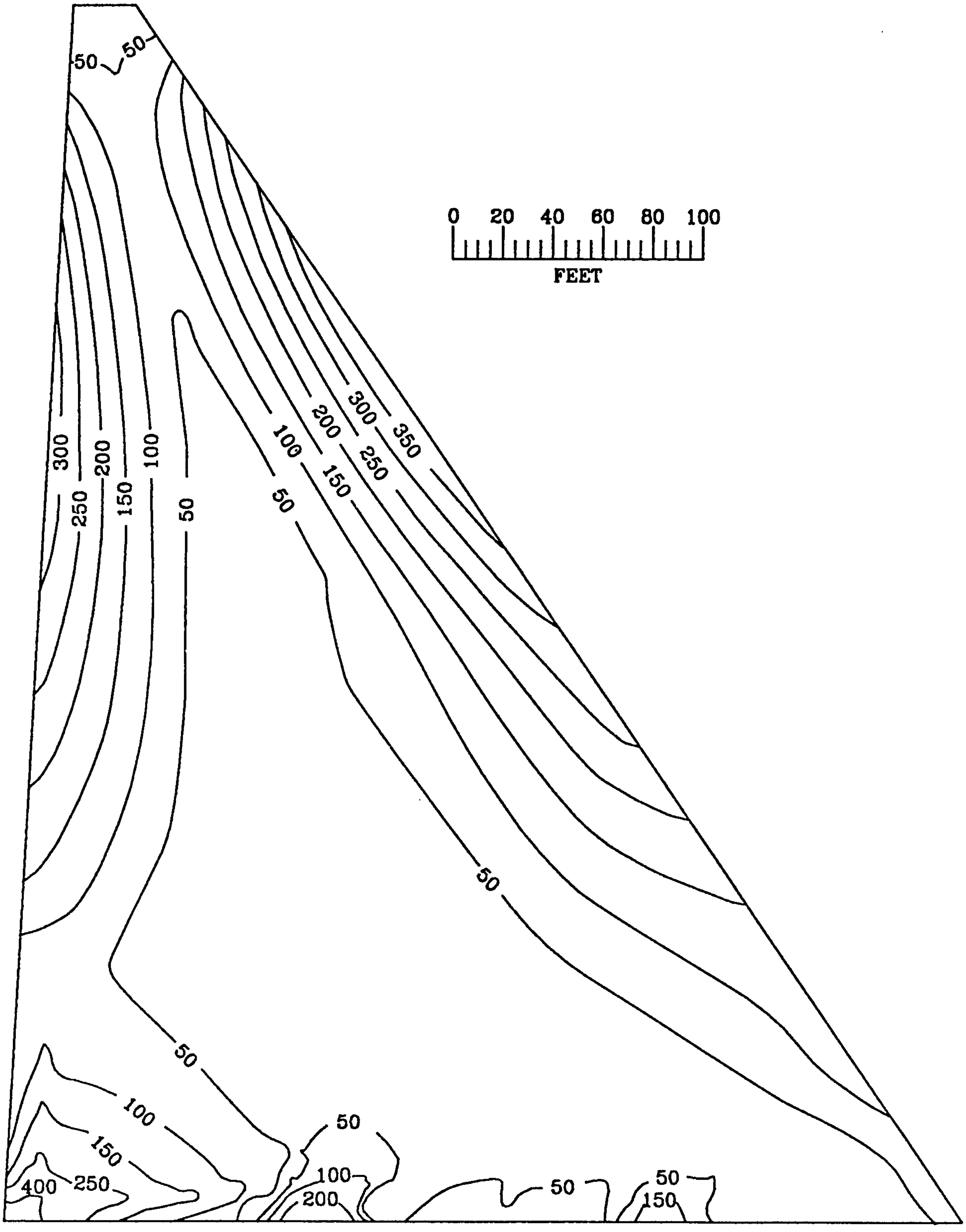
ENVELOPE VALUES OF MAXIMUM PRINCIPAL STRESSES (IN PSI) IN THE PROPOSED FLOOD CONTROL AUBURN DAM WITH POOL ELEVATION = 868.5 FEET. INITIAL STATIC STRESSES ARE INCLUDED. E_f = 2500 KSI, E_c = 3900 KSI

FIGURE 23

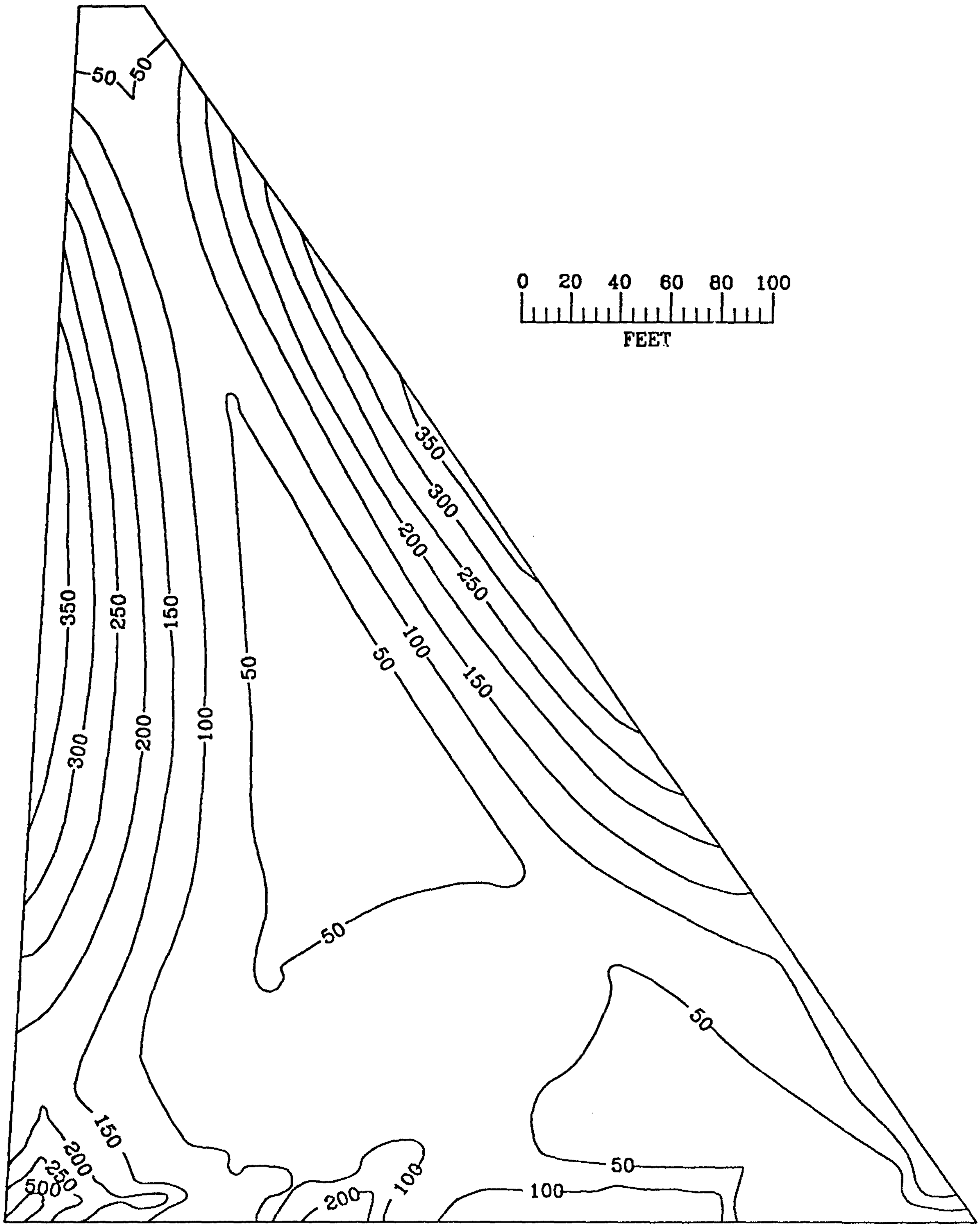


ENVELOPE VALUES OF MAXIMUM PRINCIPAL STRESSES (IN PSI) IN THE PROPOSED FLOOD CONTROL AUBURN DAM WITH POOL ELEVATION = 868.5 FEET. INITIAL STATIC STRESSES ARE INCLUDED. E_f = 2500 KSI, E_c = 4900 KSI

FIGURE 24

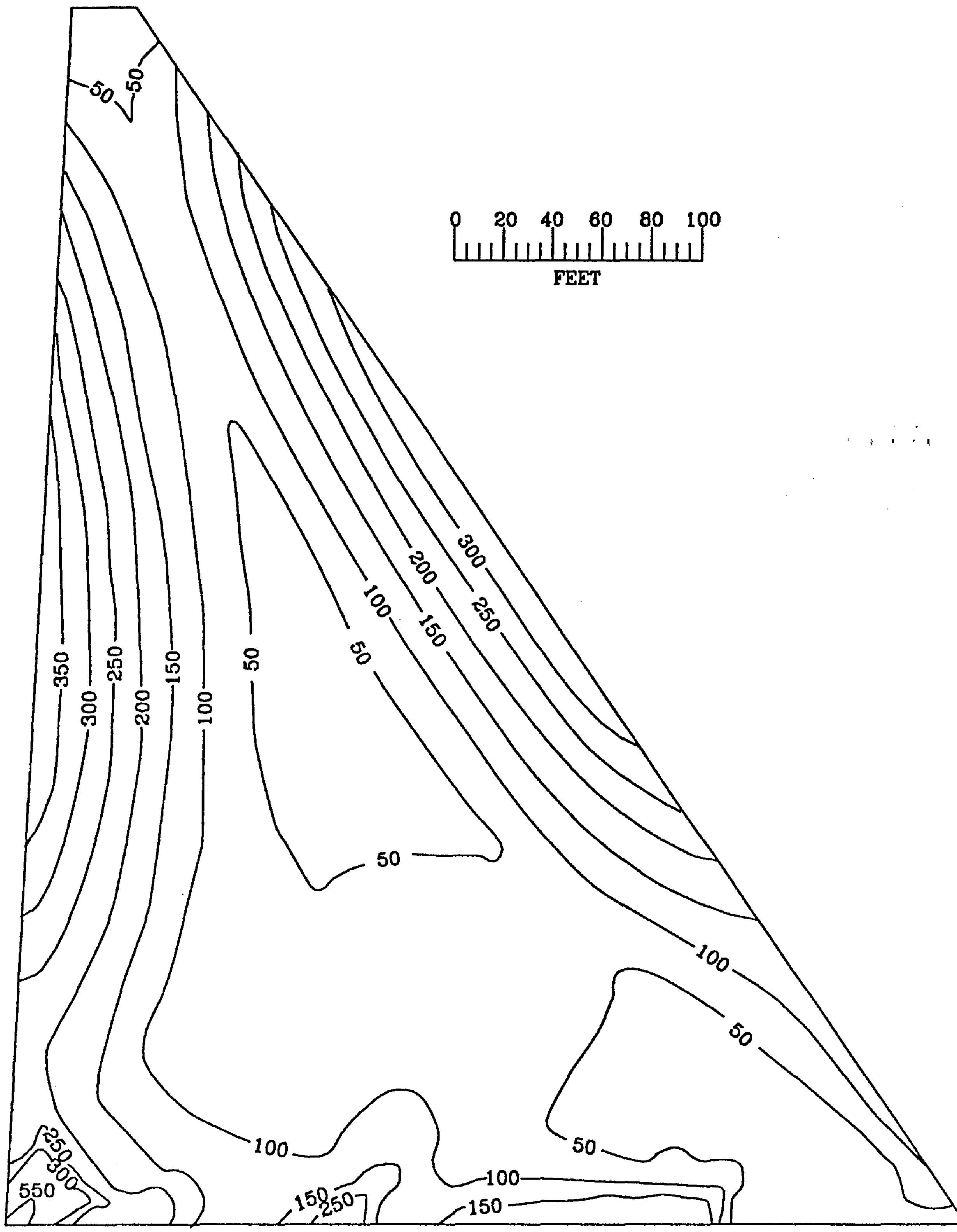


ENVELOPE VALUES OF MAXIMUM PRINCIPAL STRESSES (IN PSI) IN THE PROPOSED FLOOD CONTROL AUBURN DAM WITH POOL ELEVATION 868.5 FEET. INITIAL STATIC STRESSES ARE INCLUDED. E_f = 3750 KSI, E_c =2000 KSI



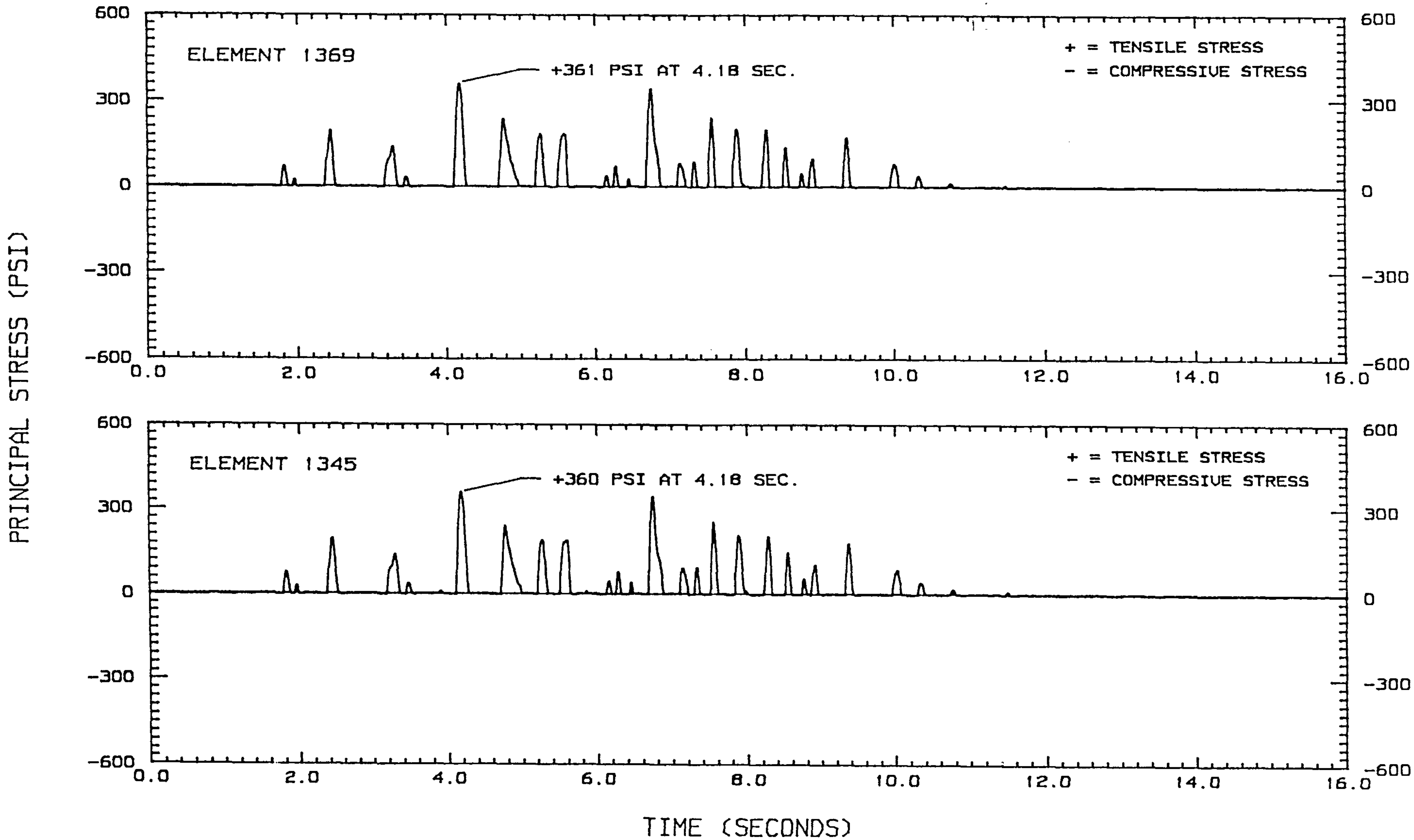
ENVELOPE VALUES OF MAXIMUM PRINCIPAL STRESSES (IN PSI) IN THE PROPOSED FLOOD CONTROL AUBURN DAM WITH POOL ELEVATION = 868.5 FEET. INITIAL STATIC STRESSES ARE INCLUDED. E_f = 3750 KSI, E_c = 3900 KSI

FIGURE 26



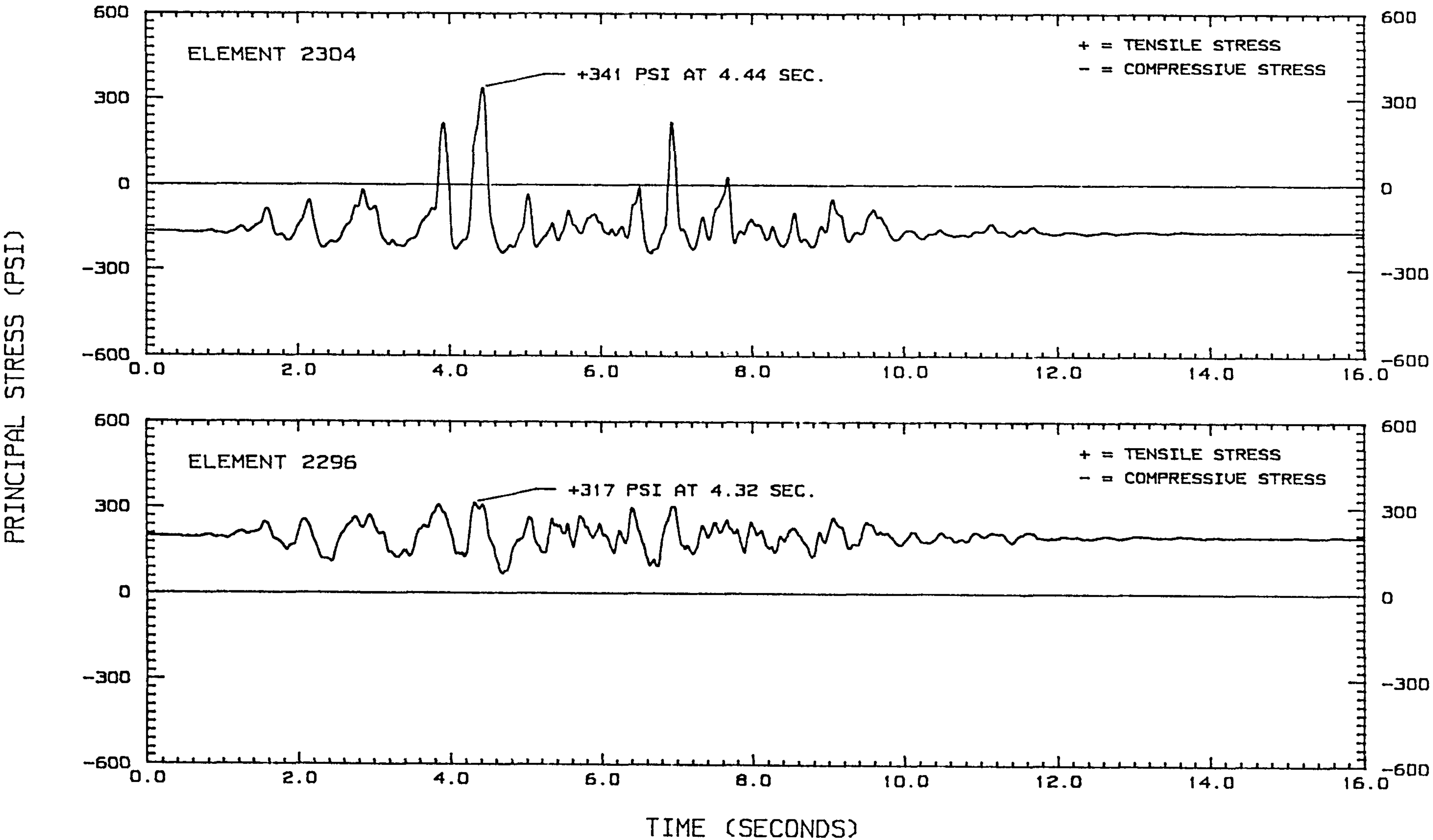
ENVELOPE VALUES OF MAXIMUM PRINCIPAL STRESSES (IN PSI) IN THE PROPOSED FLOOD CONTROL AUBURN DAM WITH POOL ELEVATION = 868.5 FEET. INITIAL STATIC STRESSES ARE INCLUDED. E_f = 3750 KSI, E_c = 4900 KSI

FIGURE 27



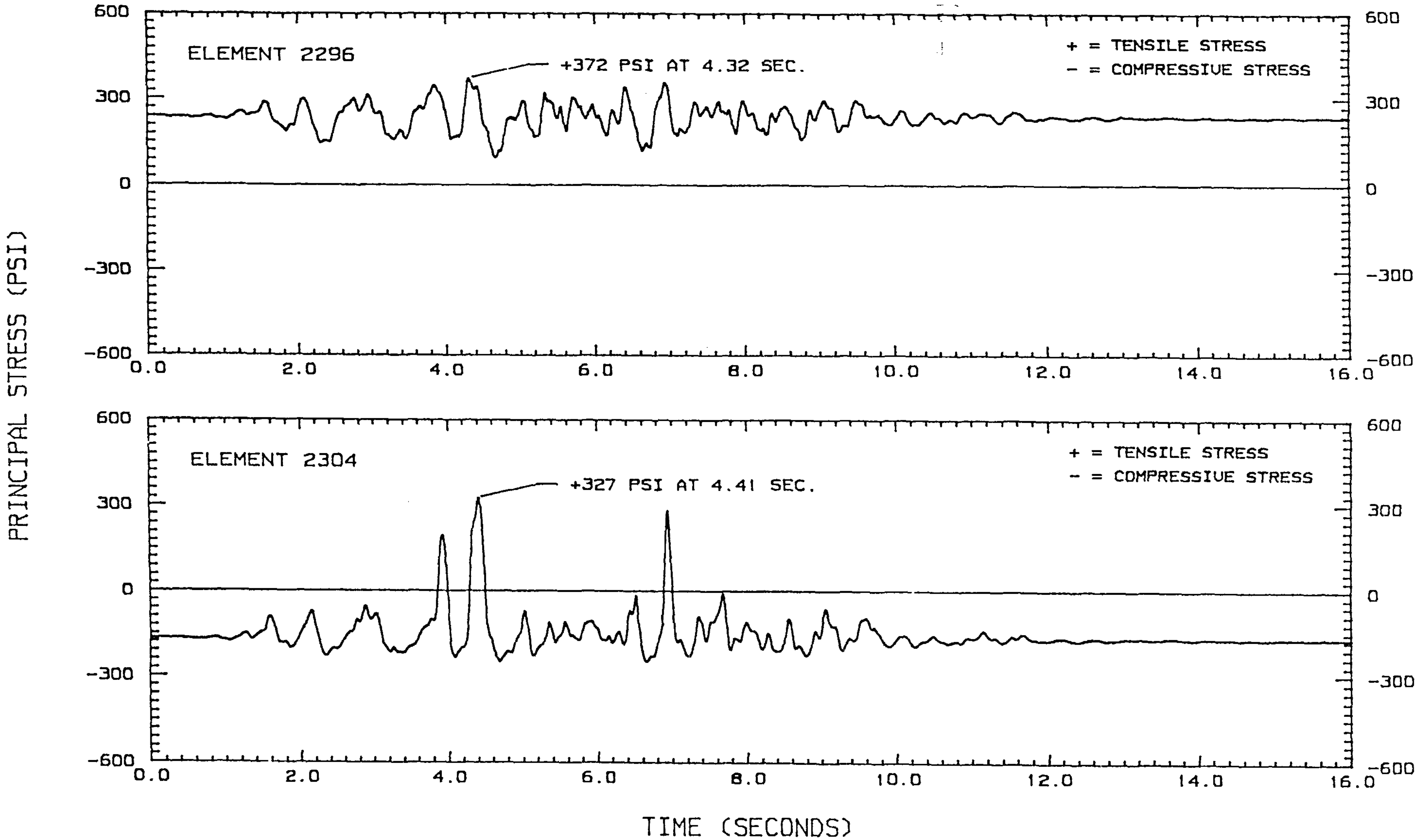
STRESS RESPONSE AT SELECTED LOCATIONS OF THE PROPOSED AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.
Ef = 2500 KSI   Ec = 2000 KSI

FIGURE 28



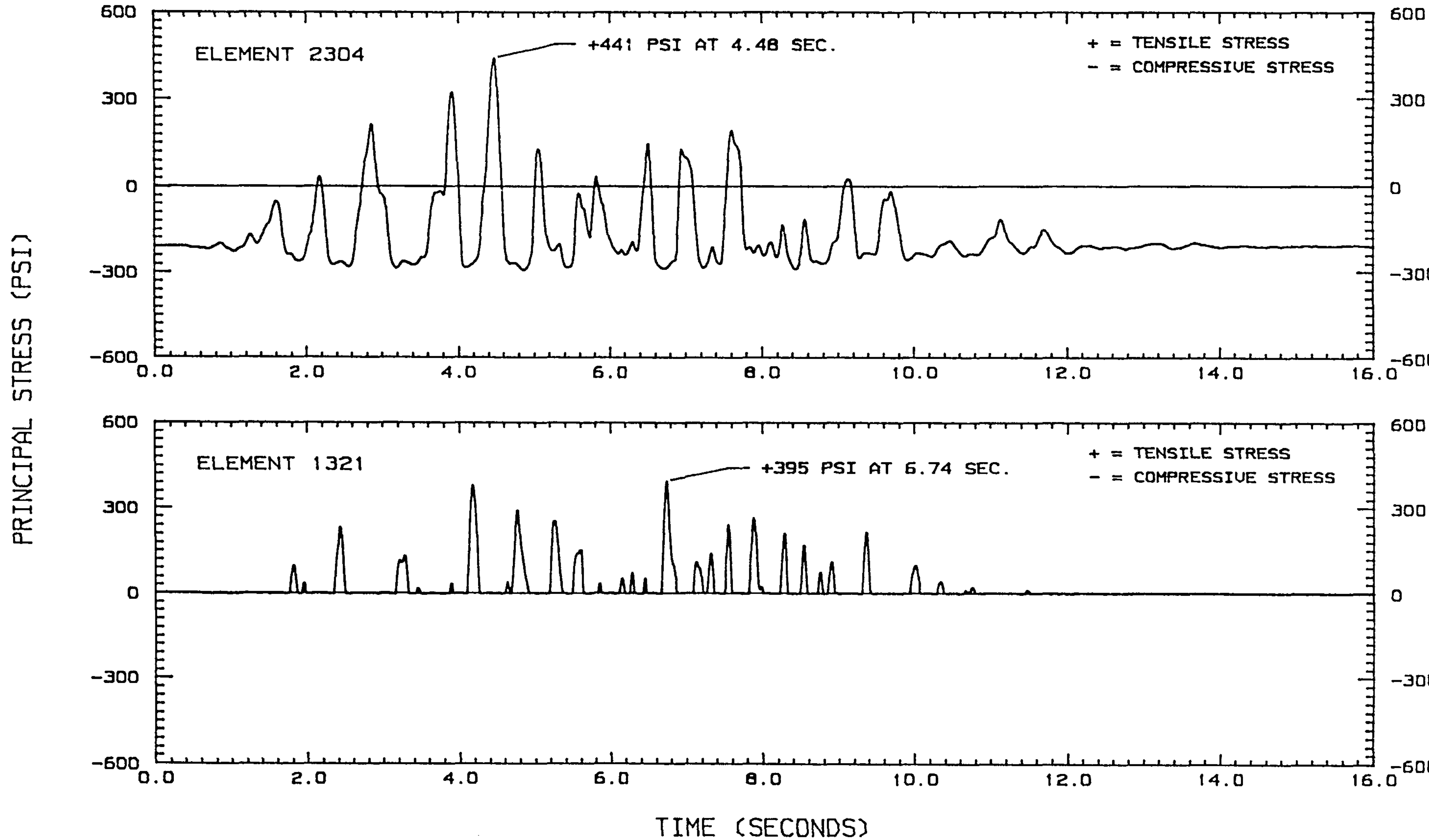
STRESS RESPONSE AT SELECTED LOCATIONS OF THE PROPOSED
AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.
Ef = 2500 KSI Ec = 3900 KSI

FIGURE 29



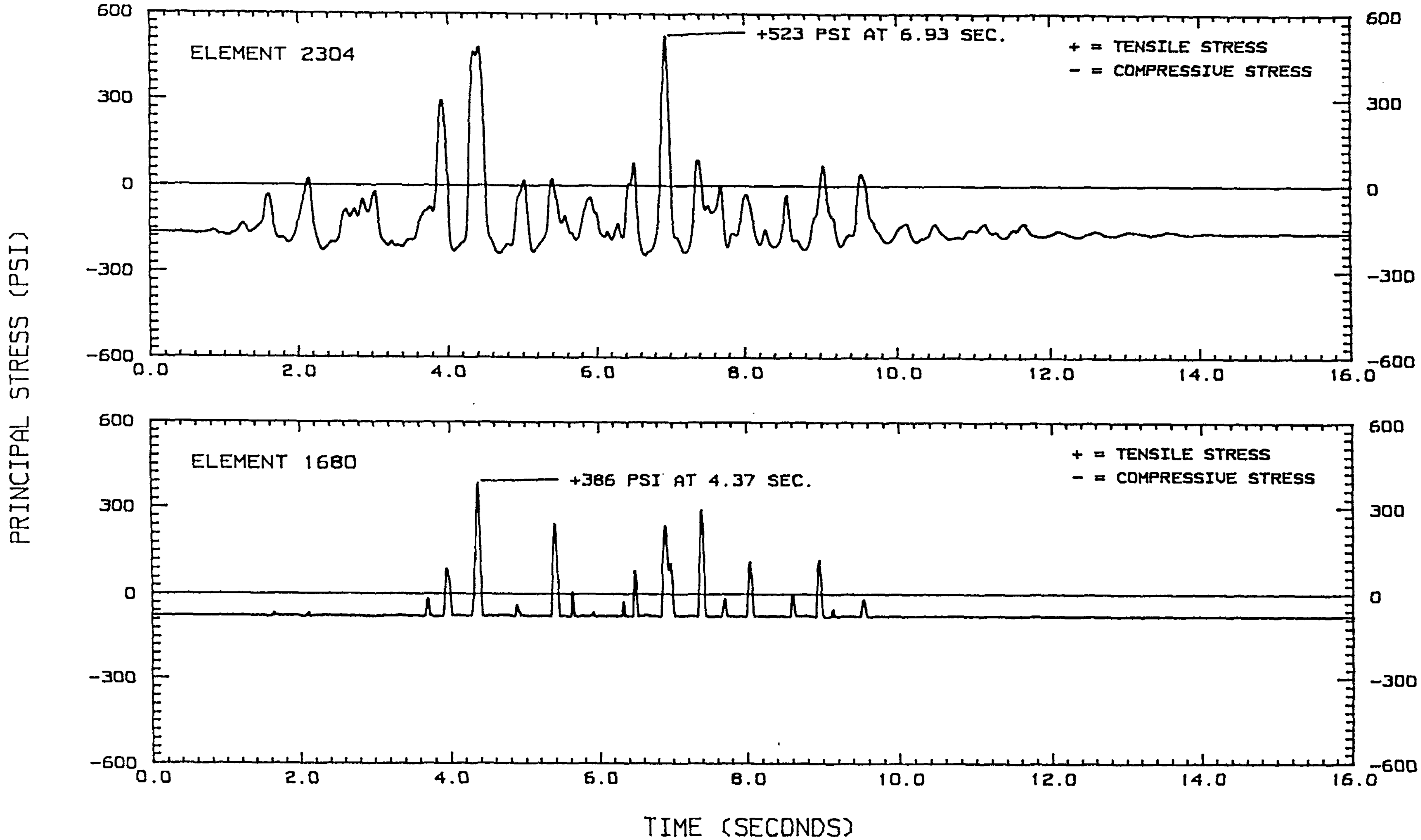
STRESS RESPONSE AT SELECTED LOCATIONS OF THE PROPOSED
AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.
E_f = 2500 KSI E_c = 4900 KSI

FIGURE 30



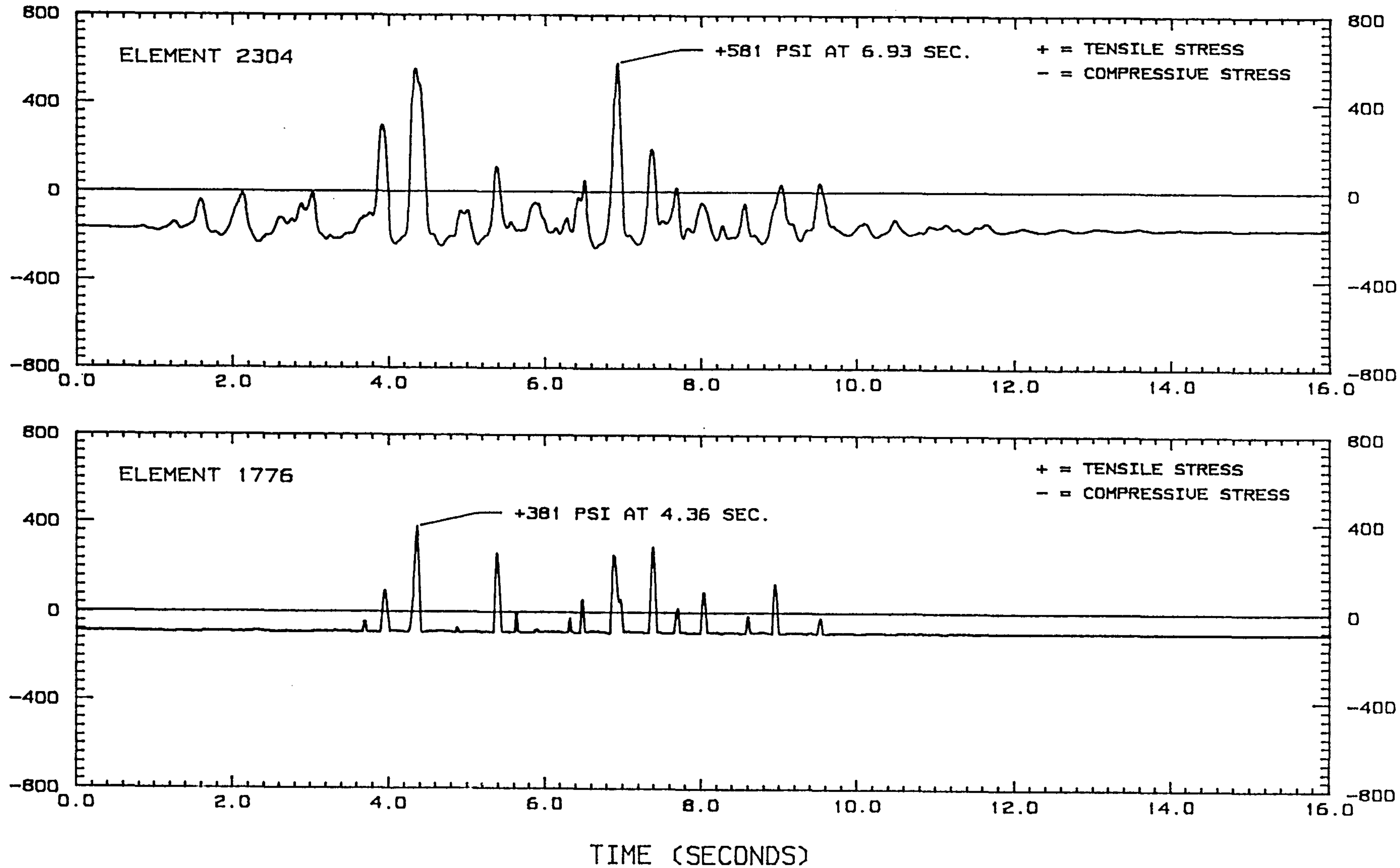
STRESS RESPONSE AT SELECTED LOCATIONS OF THE PROPOSED
AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.
E_f = 3750 KSI E_c = 2000 KSI

FIGURE 31



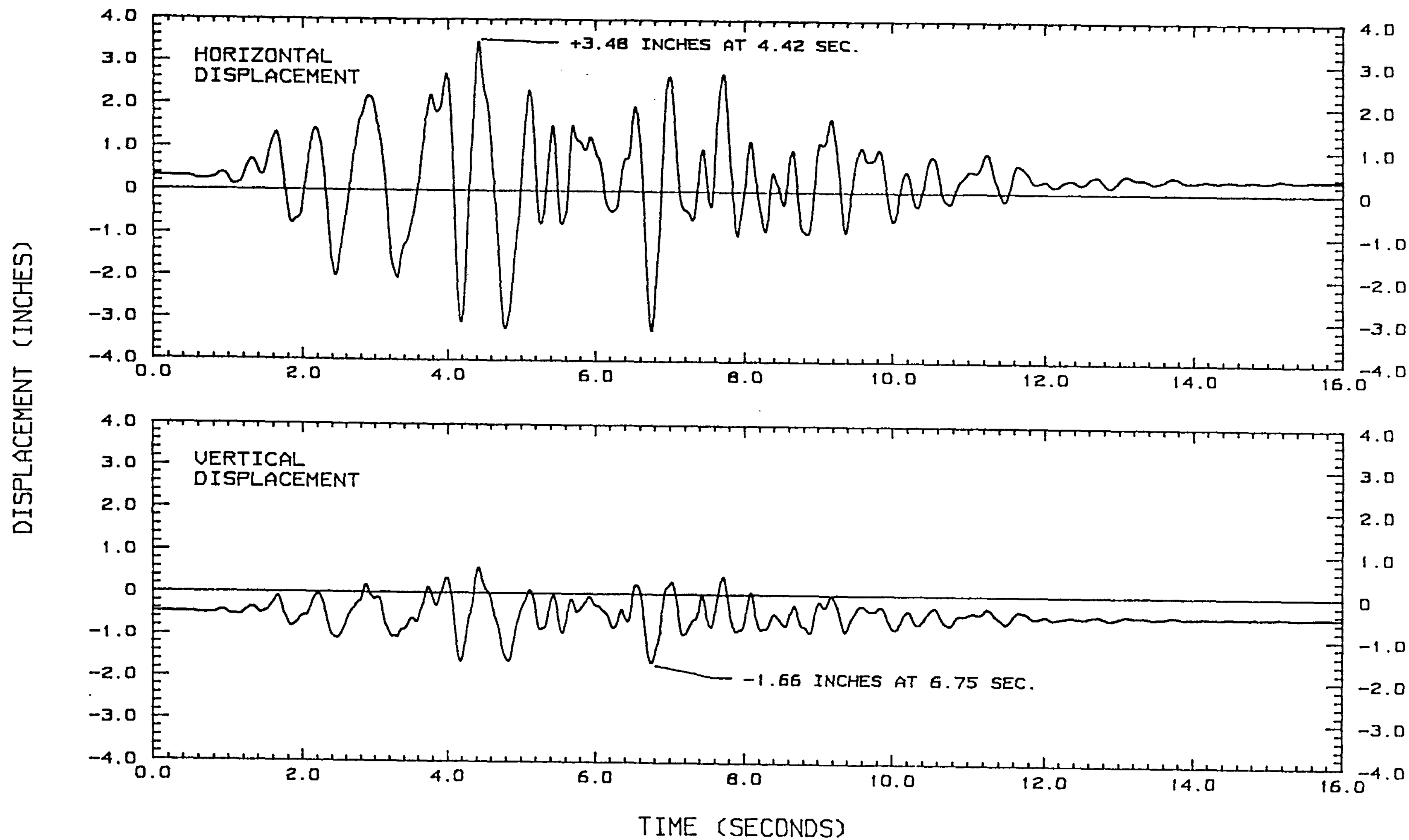
STRESS RESPONSE AT SELECTED LOCATIONS OF THE PROPOSED AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.
Ef = 3750 KSI   Ec = 3900 KSI

FIGURE 32



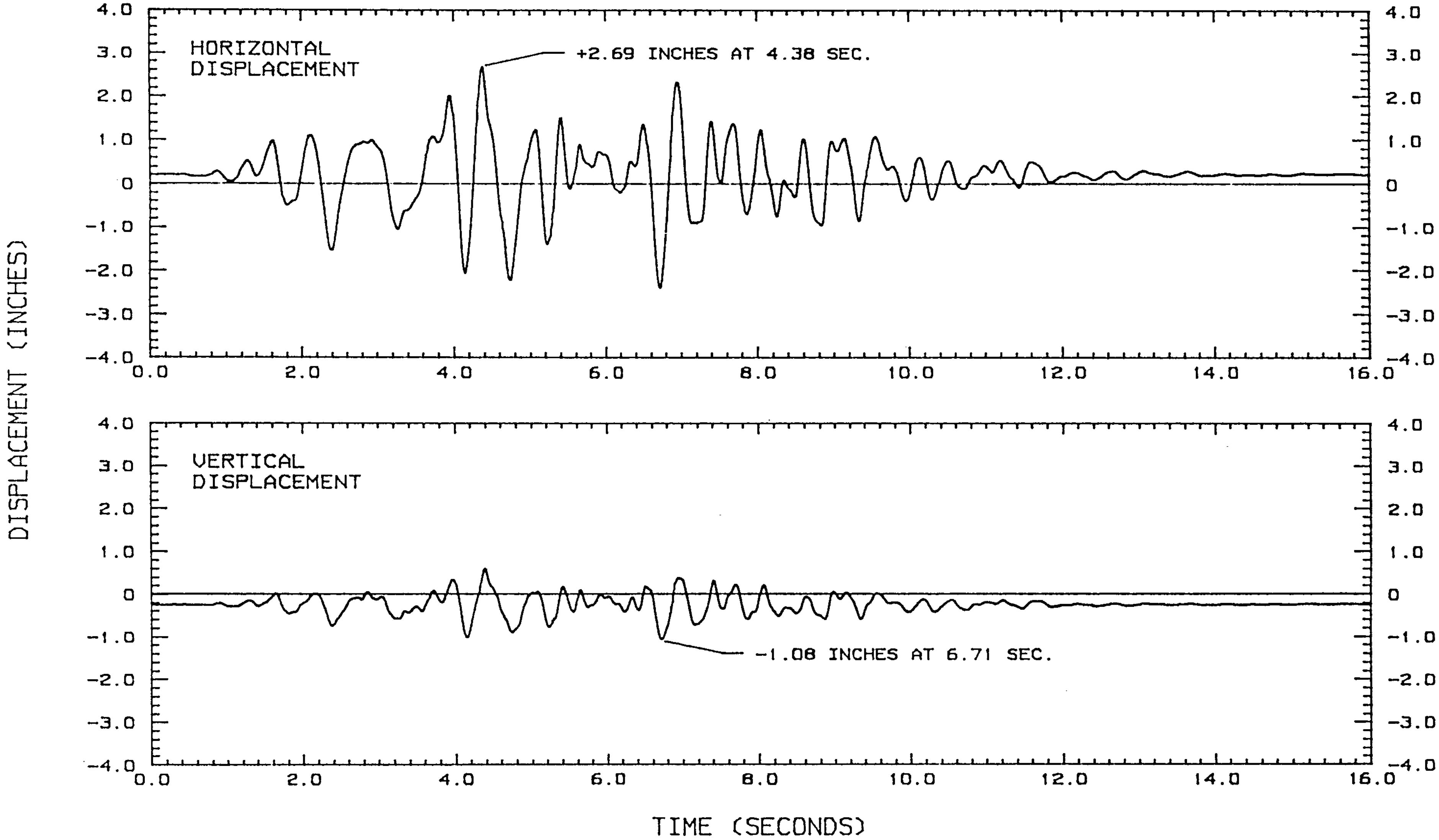
STRESS RESPONSE AT SELECTED LOCATIONS OF THE PROPOSED AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.
Ef = 3750 KSI   Ec = 4900 KSI

FIGURE 33



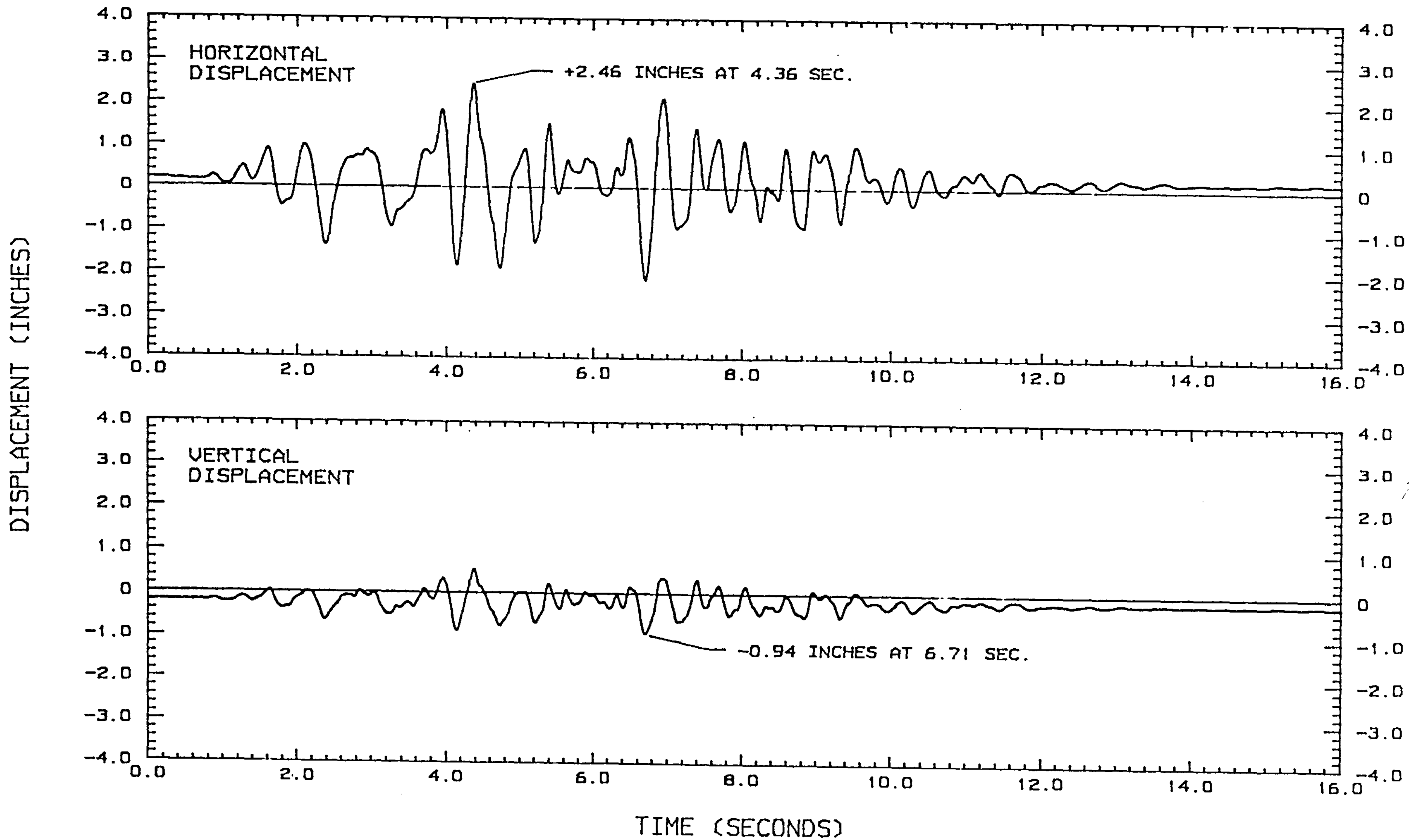
DISPLACEMENT RESPONSE AT DAM CREST OF THE PROPOSED
AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.
Ef = 2500 KSI   Ec = 2000 KSI

FIGURE 34



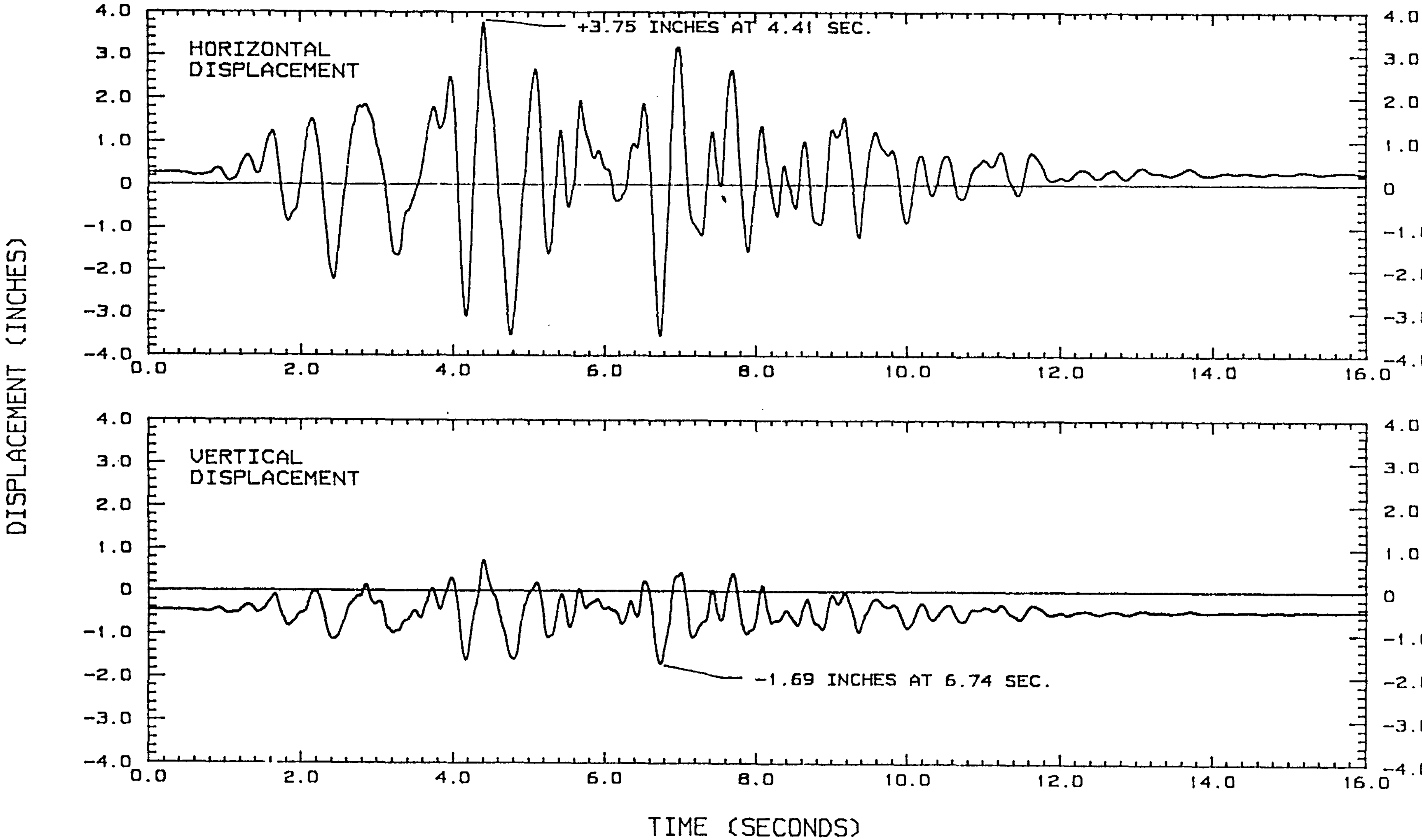
DISPLACEMENT RESPONSE AT DAM CREST OF THE PROPOSED
AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.
Ef = 2500 KSI   Ec = 3900 KSI

FIGURE 35



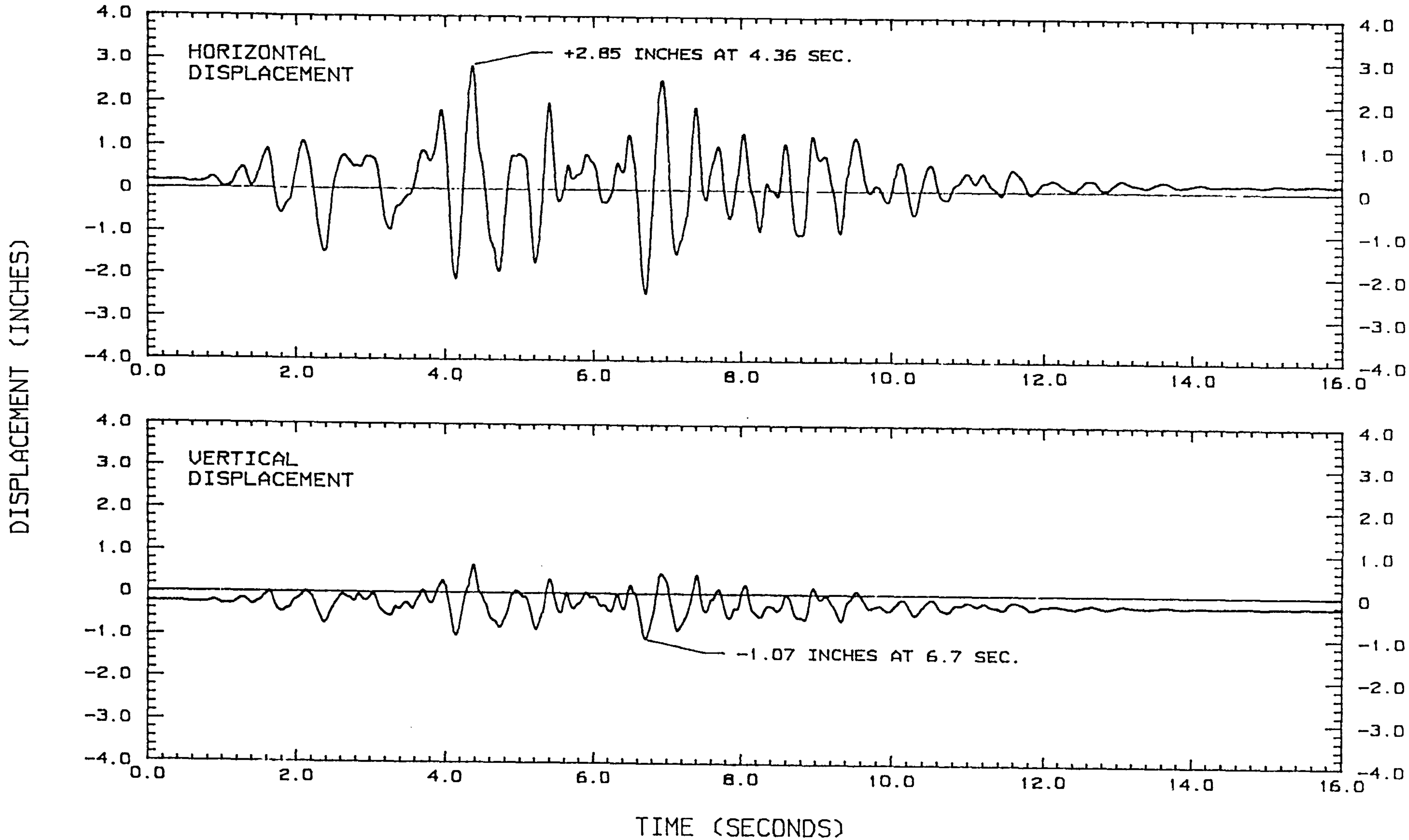
DISPLACEMENT RESPONSE AT DAM CREST OF THE PROPOSED
AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.
Ef = 2500 KSI   Ec = 4900 KSI

FIGURE 36



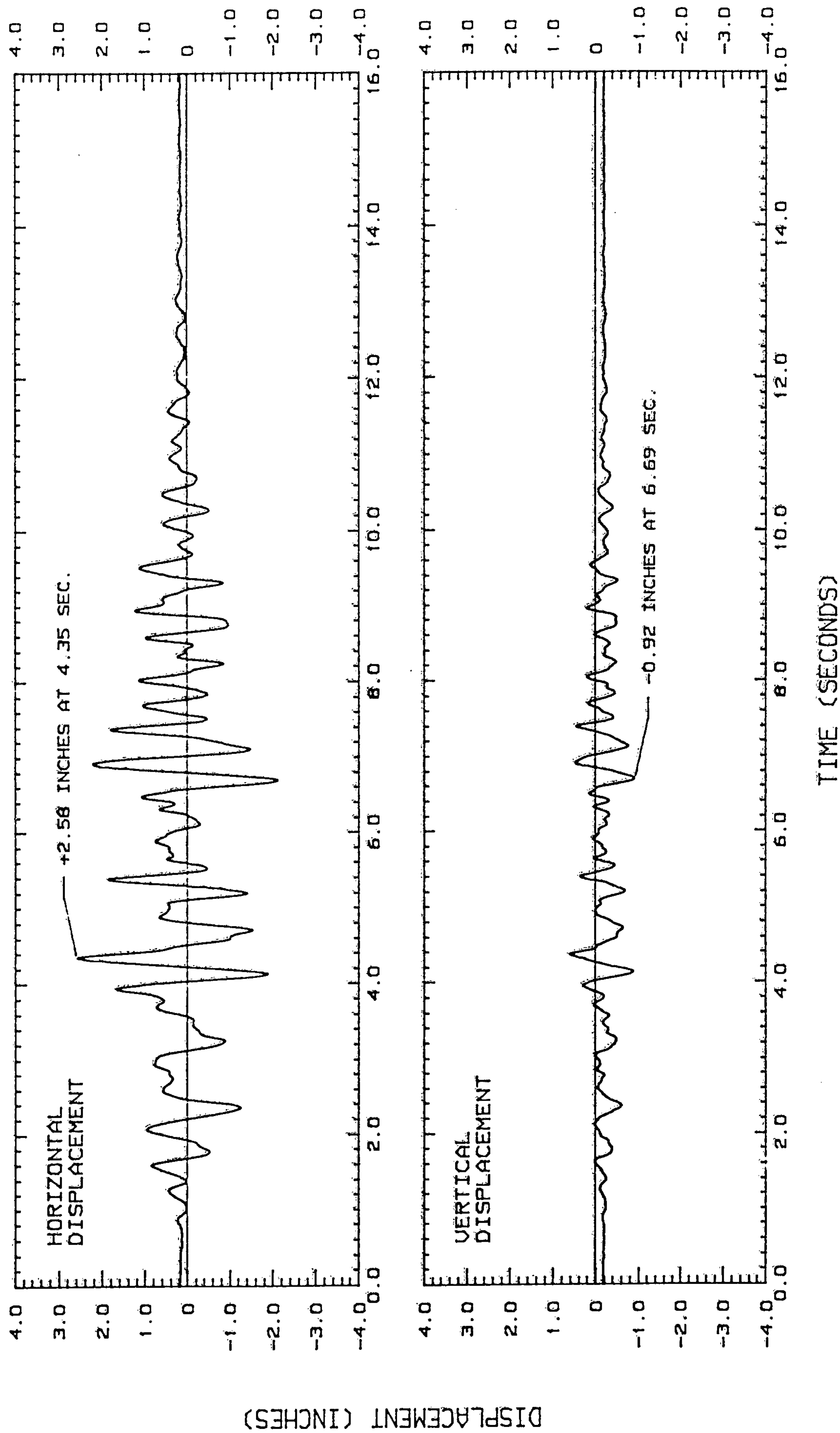
DISPLACEMENT RESPONSE AT DAM CREST OF THE PROPOSED
AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.
E_f = 3750 KSI E_c = 2000 KSI

FIGURE 37



DISPLACEMENT RESPONSE AT DAM CREST OF THE PROPOSED
AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.
Ef = 3750 KSI   Ec = 3900 KSI

FIGURE 38



DISPLACEMENT RESPONSE AT DAM CREST OF THE PROPOSED
AUBURN DAM WITH POOL ELEVATION = 868.5 FEET.
Ef = 3750 KSI   Ec = 4900 KSI

FIGURE 39

For all cases with different combinations of Ec and Ef, the maximum tensile stresses are below the assumed apparent dynamic tensile strengths (see Table N-3-29). Thus, cracking of concrete would not occur under earthquake conditions. On the basis of the overall stress levels, it can be expected that the dam response is predominantly linearly elastic, consistent with the assumption of the finite element analysis.

In conclusion, the selected plan flood control dam is capable of withstanding the maximum credible earthquake and post-earthquake loads (hydrostatic pressure and dead weight of dam) in such a way that no failure triggering a sudden, catastrophic release of water will occur, and that life and property downstream will not be endangered.

Future Structural Studies - Structural analysis for the selected plan has been limited to a dynamic analysis based on the design seismic event. Much remains to be done in the detailed design phase. Future detailed design efforts for the concrete gravity dam (RCC) will include, as a minimum, the following:

1. Further analysis of the curved alignment.
2. General stability analysis of the dam cross section.
3. Improved estimates of strength parameters for RCC.
4. Detailed foundation investigation and foundation treatment requirements from geotechnical studies.
5. Investigation of load cases other than the MCE, such as normal cases and the Operational Basis Earthquake (OBE).
6. Parametric finite element studies.
7. Finite element analysis using a different computer program for verification of results.
8. A study of stresses in the dam due to fault displacements.
9. A thermal loading analysis.

Cost Estimate

The Cost Estimate for the selected plan is provided in Chapter 4 of this appendix.

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13. G. Fenves and A.K. Chopra, "Simplified Analysis for Earthquake Resistant Design of Concrete Gravity Dams," Report No. UCB/EERC-85/10, Earthquake Engineering Research Center, University of California, Berkeley, June 1986.

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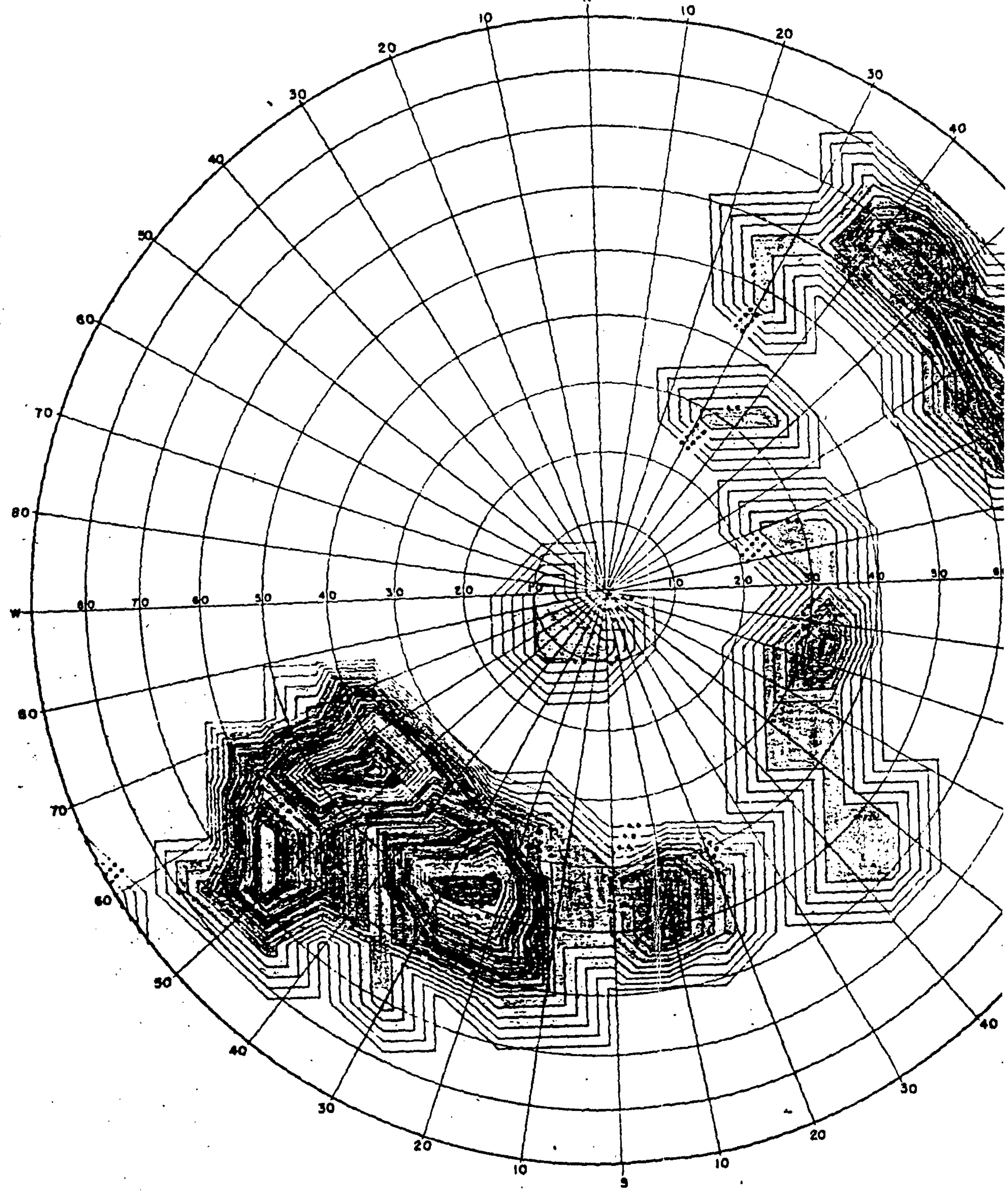
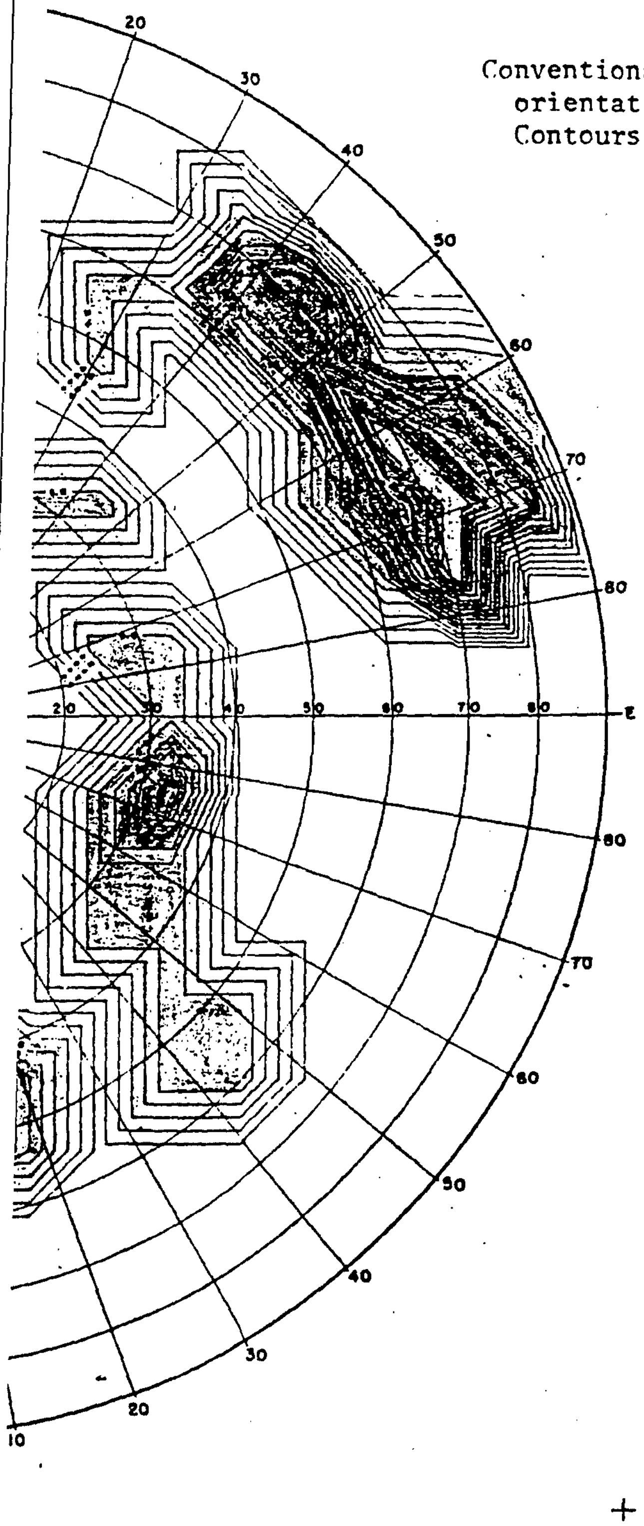
APPENDIX N-3-A

SHEAR DIAGRAMS



AUBURN DAM TUNNEL NO 2 SHEARS
04/15/68



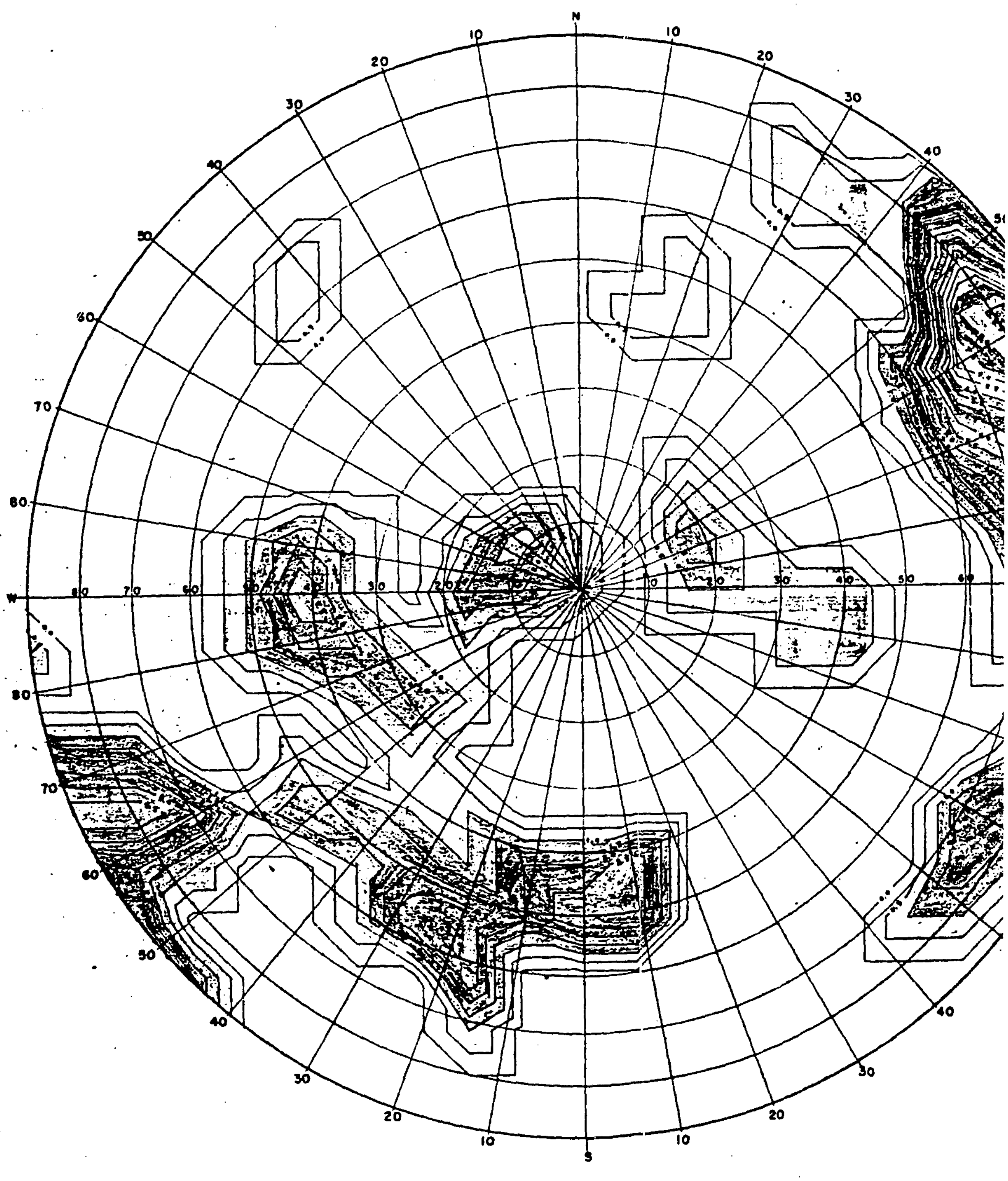
Conventional diagram--no correction for orientation or length of traverse. Contours represent number of shears.

ALWAYS THINK SAFETY

TUNNEL #2
SHEARS

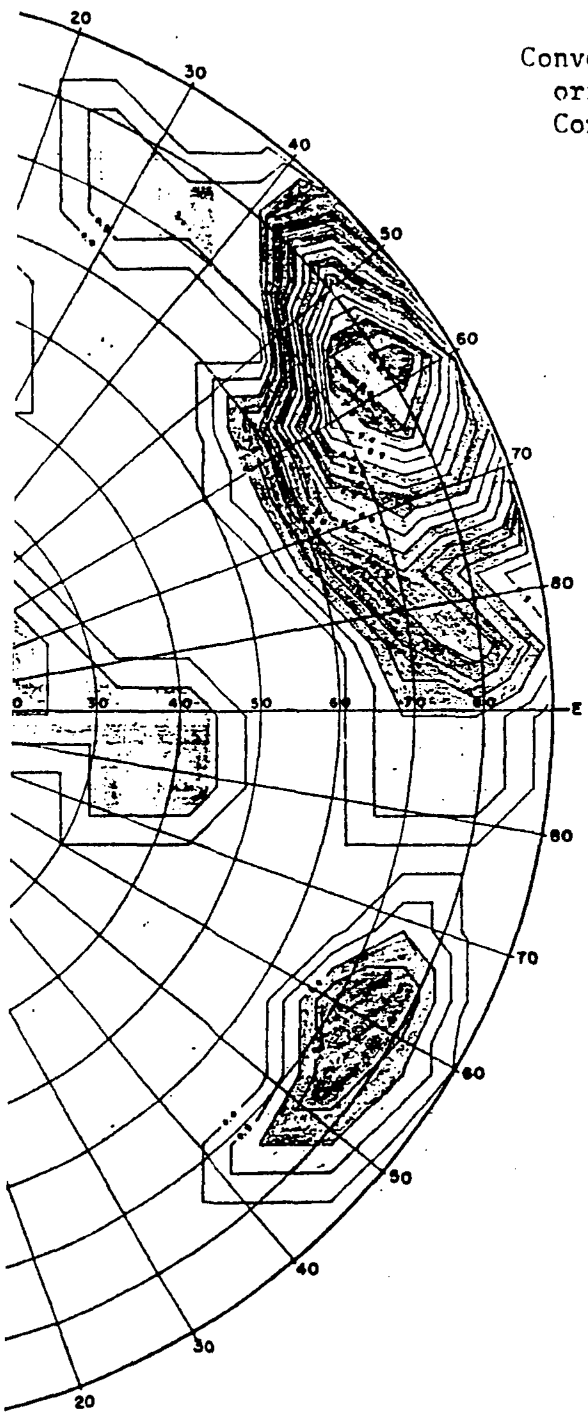
DRAWN ________ SUBMITTED ________
TRACED ________ RECOMMENDED ________
CHECKED ________ APPROVED ________
DENVER, COLORADO

1 of 10



AUBURN DAM TUNNEL NO 4 SHEARS
04/15/68

Conventional diagram--no correction for
orientation or length of traverse.
Contours represent number of shears.

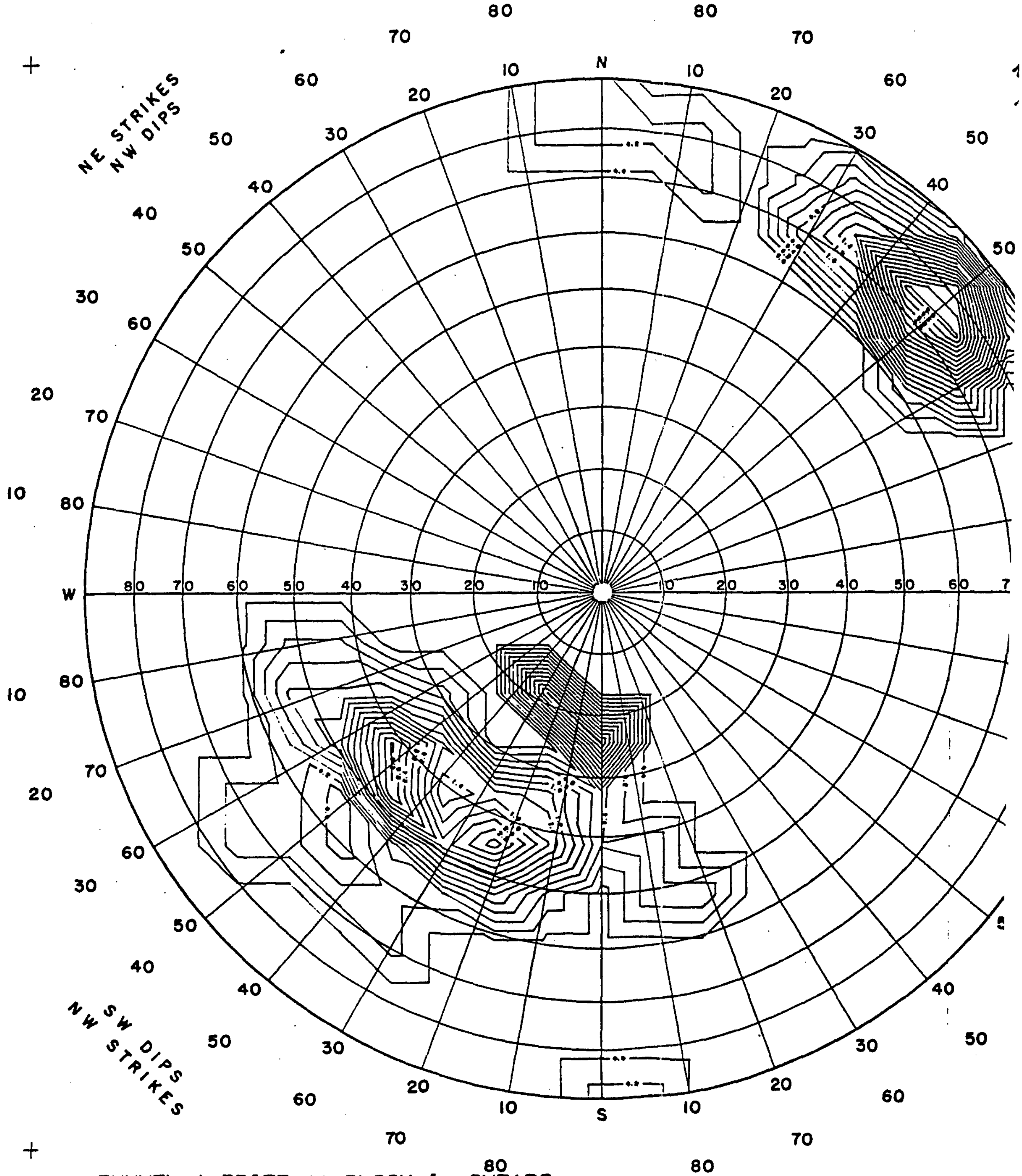


ALWAYS THINK SAFETY

Tunnel #4
SHEARS

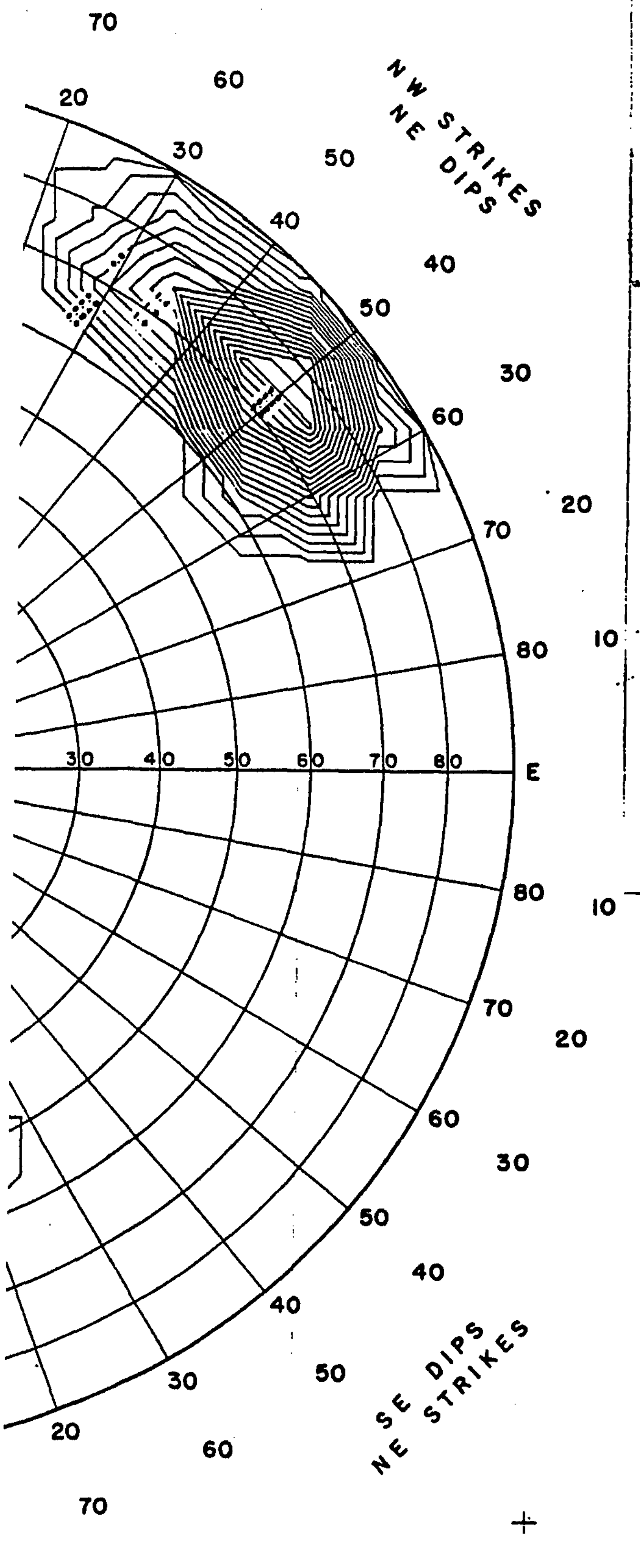
DRAWN
SUBMITTED
TRACED
RECOMMENDED
CHECKED
APPROVED
DENVER, COLORADO

2



TUNNEL 1, DRIFT 1A, BLOCK I, SHEARS.
07/15/69

J.W.J.



EXPLANATION

Countoured Shear Density Diagram
Equal-Area Projection
Upper Hemisphere

Measurements of true strike and dip of shears are made along a linear traverse, and are plotted using Terzaghi 1/ corrections and hole-length factors. Each shear plane is plotted at the equal-area projection of the intersection of its normal with a reference (upper) hemisphere. The value of the point for contouring purposes equals 100.0 divided by the sine of the angle of intersection of the plane and the traverse, divided by the length of the traverse along which measurements are made. Planes which intersect the traverse at an angle of less than ° (i.e. within a blind zone of °) are not computed.

Points are contoured by summing the value of all points within circles of 1% area on the hemisphere, centered on points which form a grid with a spacing 1/10 of the radius of the reference sphere. No values are computed for grid points which fall within the blind zone.

Contours represent shear frequencies in occurrences per 100 feet normal to each set of shears per 1% area. Contour densities are indicated in the boxes below.

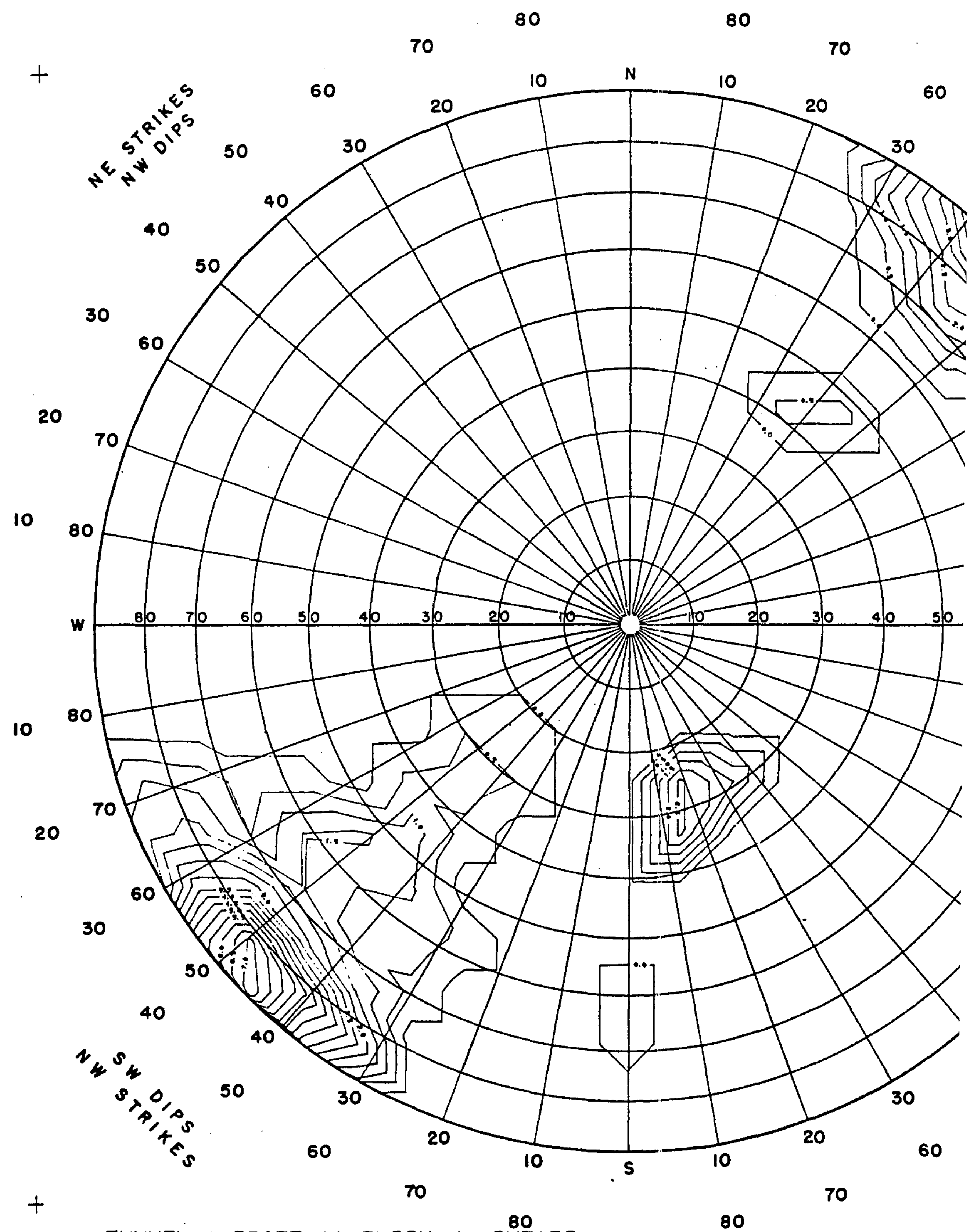
1/ Sources of Error in Joint Surveys, Ruth D. Terzaghi, Geotechnique, Vol. 15, No. 3, P. 287-304, 1965.

NOTE

Use the outer circle of numbers for reading strike azimuth, the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

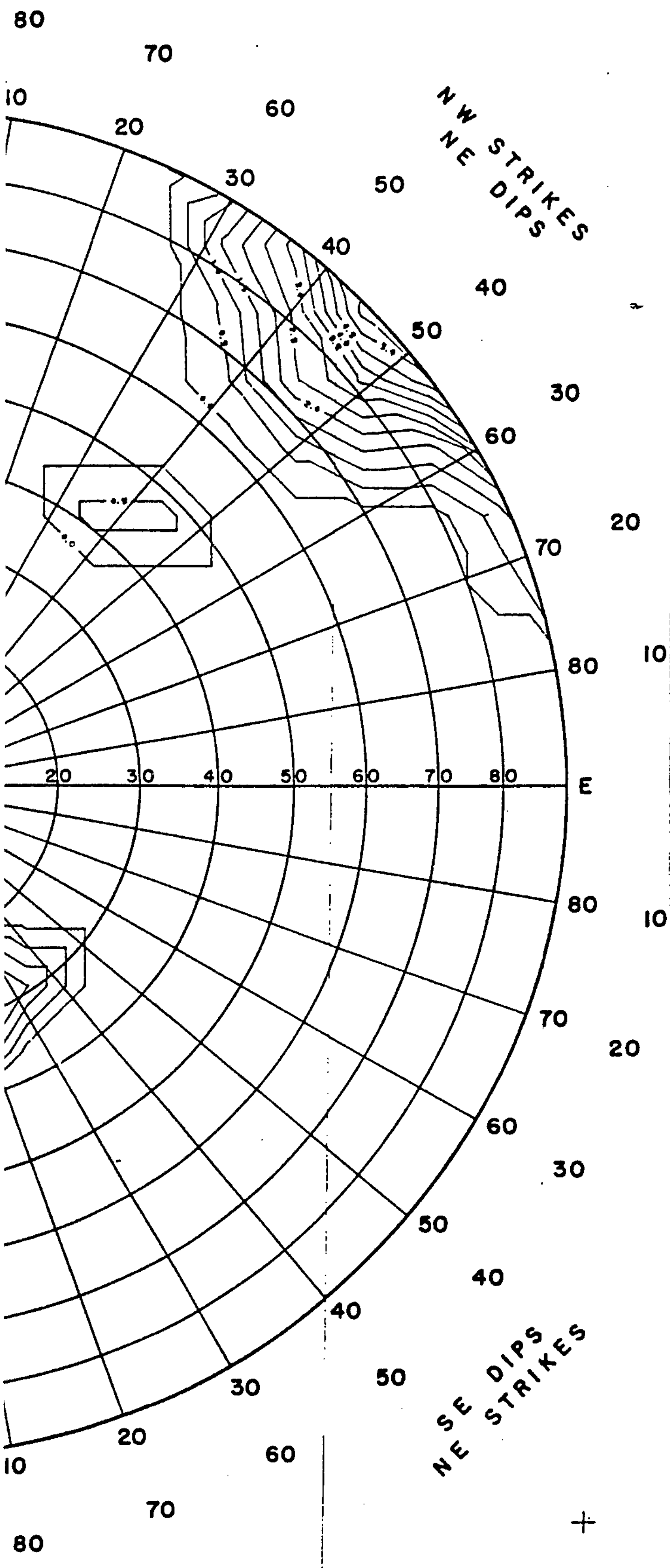
DRAWN ____ SUBMITTED ____
TRACED A.R.R. RECOMMENDED ____
CHECKED ____ APPROVED ____

2



TUNNEL 1, DRIFT 1A, BLOCK J, SHEARS
07/03/69

J.W.J.



EXPLANATION

Countoured Shear Density Diagram
Equal-Area Projection
Upper Hemisphere

Measurements of true strike and dip of shears are made along a linear traverse, and are plotted using Terzaghi 1/ corrections and hole-length factors. Each shear plane is plotted at the equal-area projection of the intersection of its normal with a reference (upper) hemisphere. The value of the point for contouring purposes equals 100.0 divided by the sine of the angle of intersection of the plane and the traverse, divided by the length of the traverse along which measurements are made. Planes which intersect the traverse at an angle of less than 5° (i.e. within a blind zone of 5°) are not computed.

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NOTE

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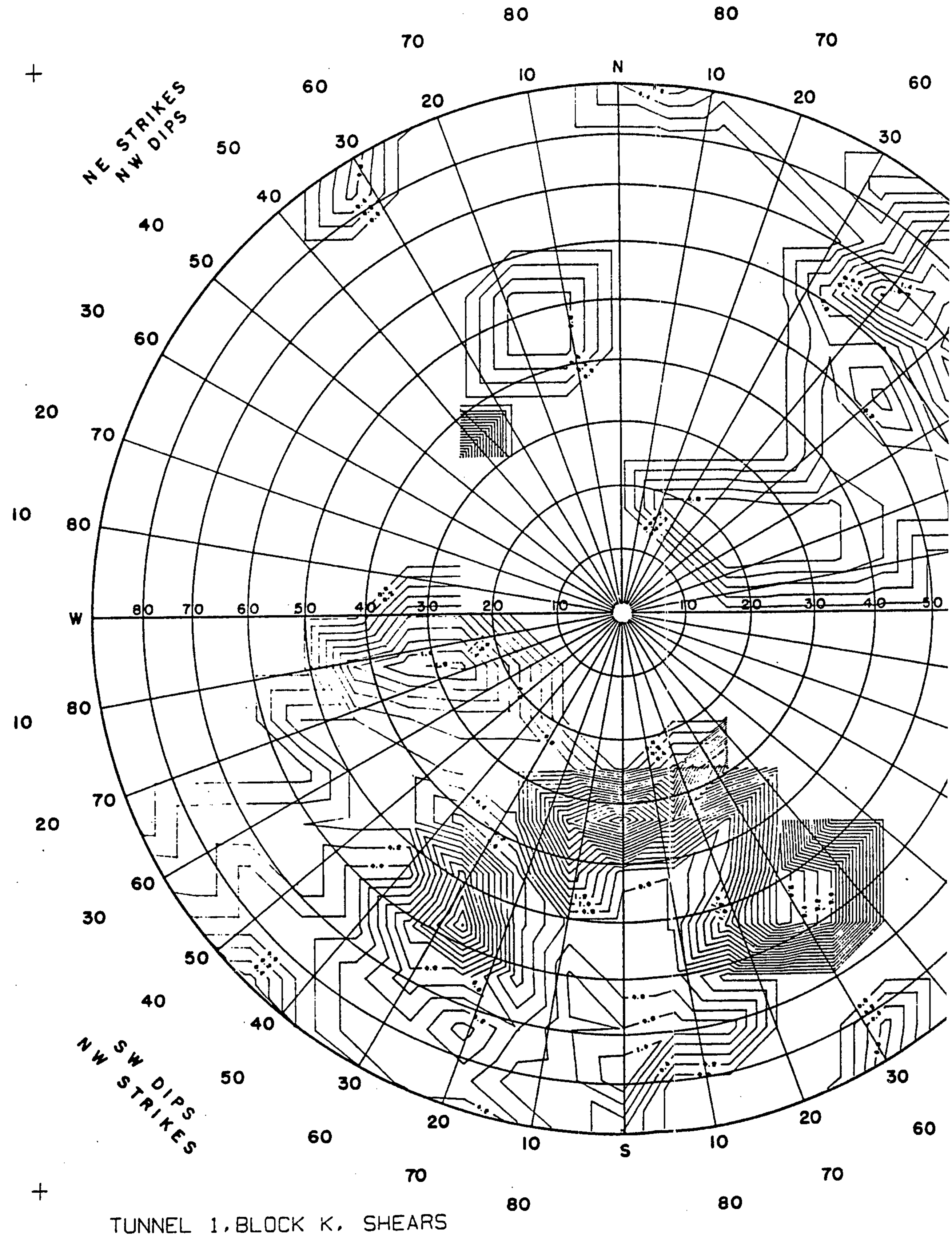
DRAWN	SUBMITTED
TRACED A.R.R.	RECOMMENDED
CHECKED	APPROVED

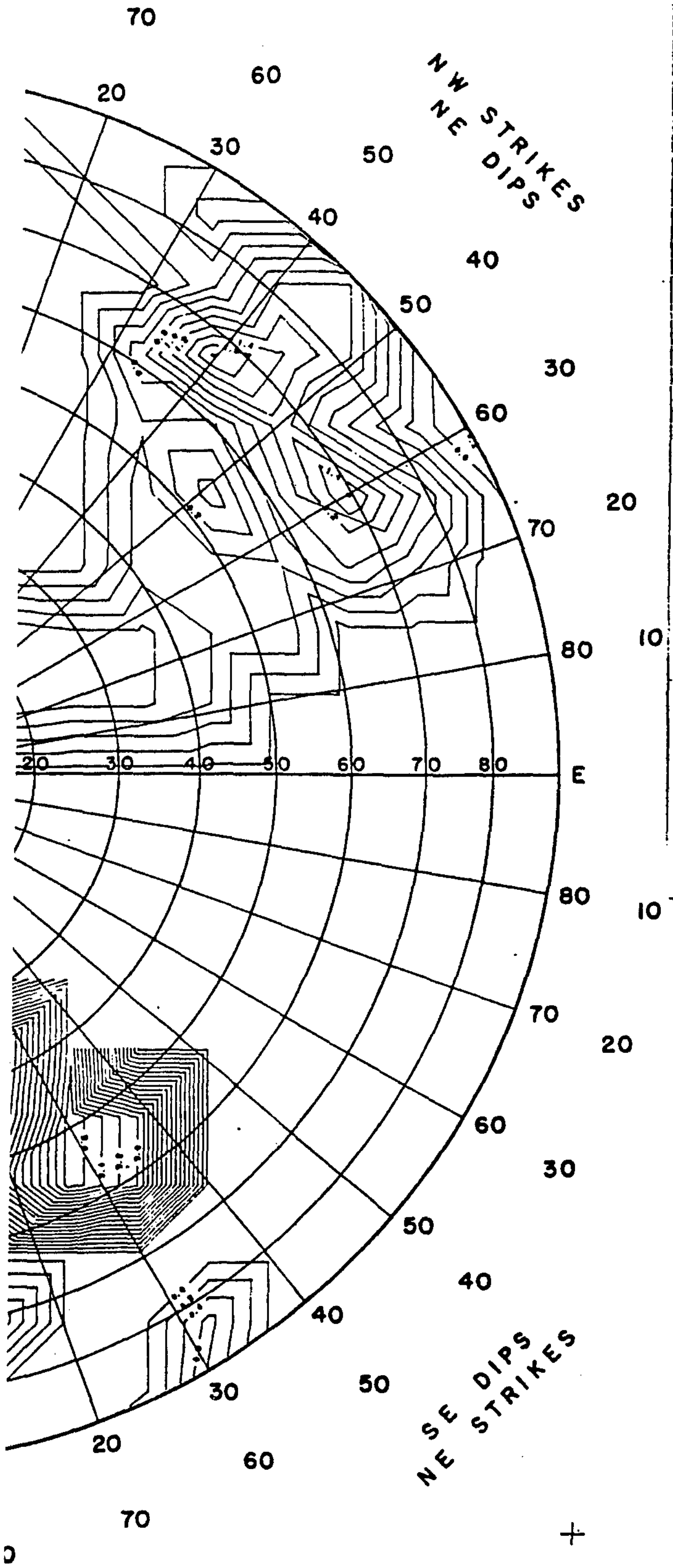
2

4 of 10



TUNNEL 1, BLOCK K, SHEARS
6 13 69

J.W.J.



EXPLANATION

Countoured Shear Density Diagram
Equal-Area Projection
Upper Hemisphere

Measurements of true strike and dip of shears are made along a linear traverse, and are plotted using Terzaghi 1/ corrections and hole-length factors. Each shear plane is plotted at the equal-area projection of the intersection of its normal with a reference (upper) hemisphere. The value of the point for contouring purposes equals 100.0 divided by the sine of the angle of intersection of the plane and the traverse, divided by the length of the traverse along which measurements are made. Planes which intersect the traverse at an angle of less than ° (i.e. within a blind zone of °) are not computed.

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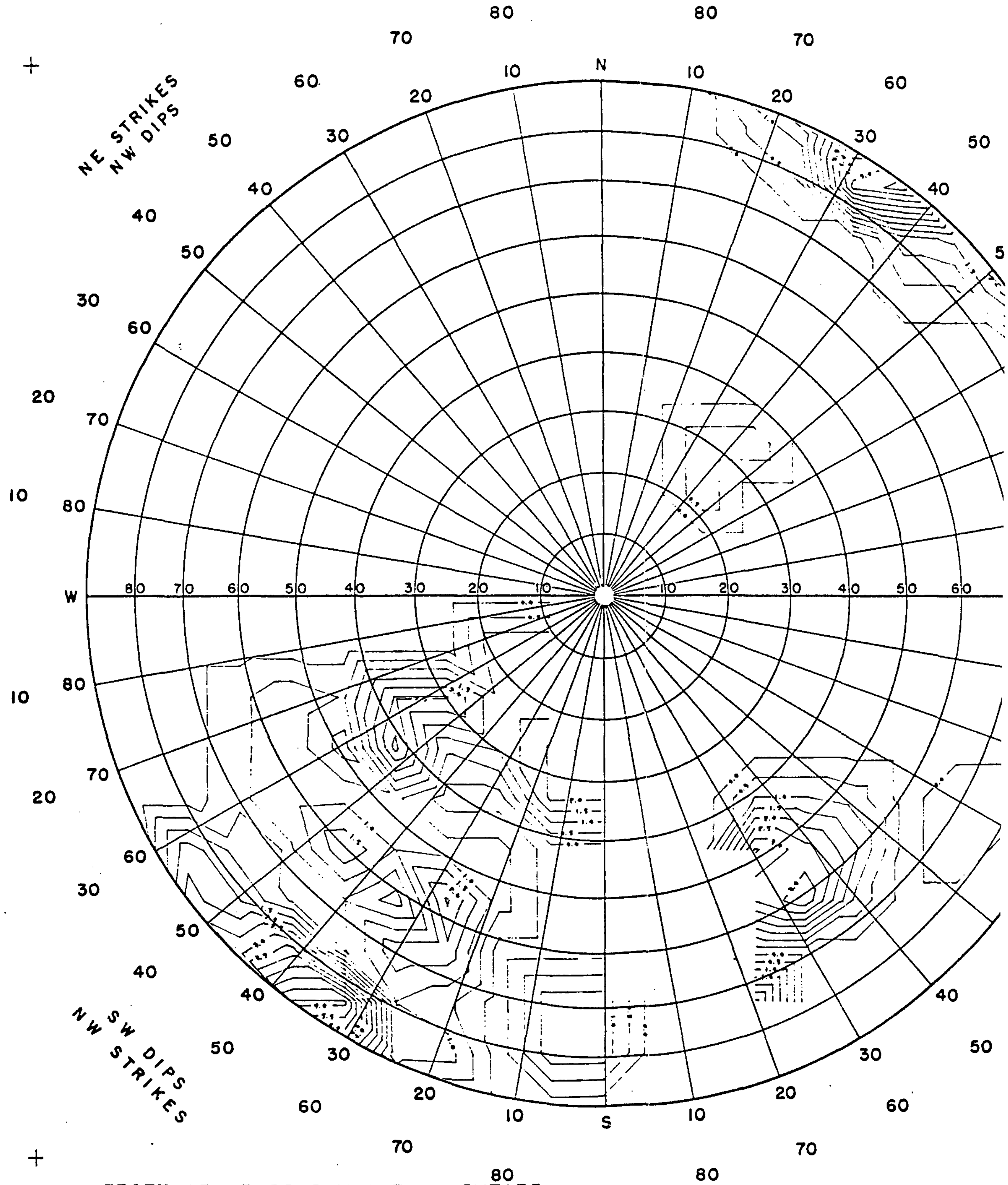
NOTE

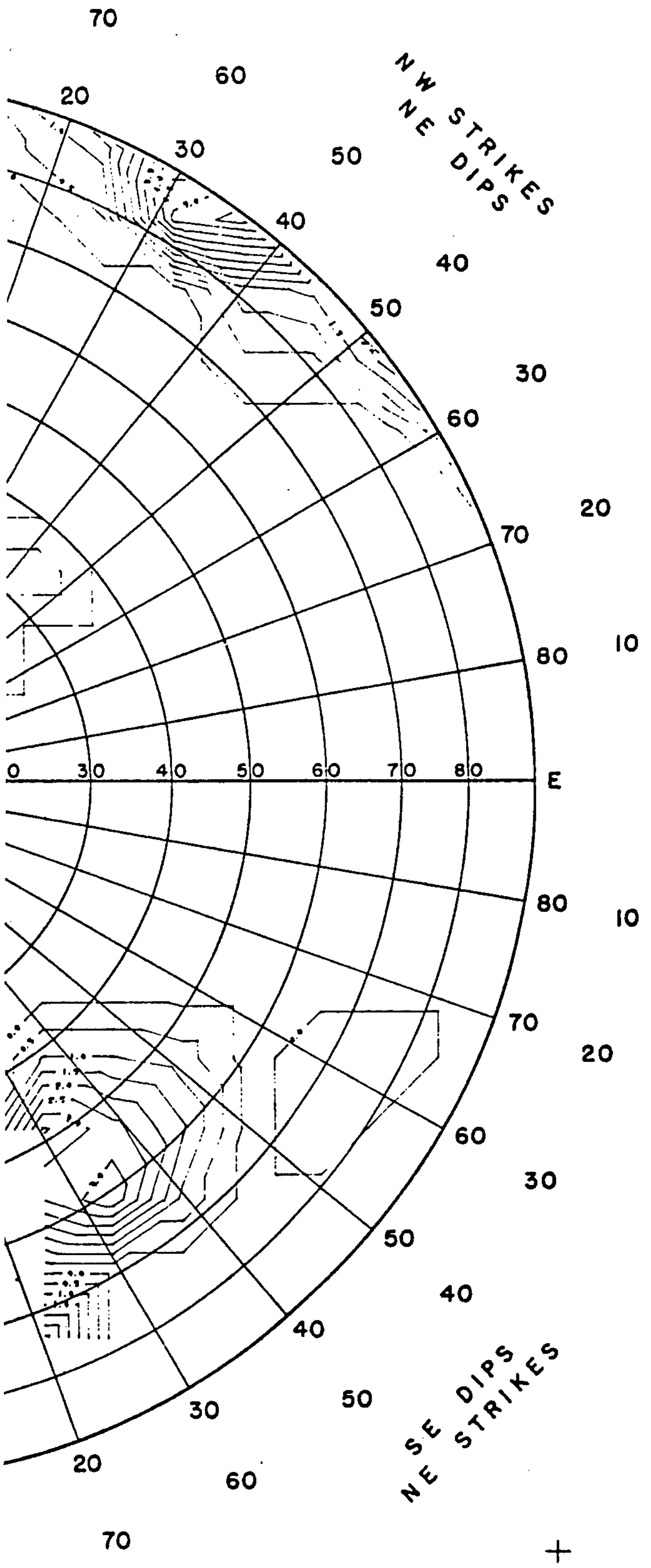
Use the outer circle of numbers for reading strike azimuth, the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

DRAWN ____________	SUBMITTED ____________
TRACED A.P.R.	RECOMMENDED ____________
CHECKED ____________	APPROVED ____________

2



DRIFT 1B, BLOCKS K AND J, SHEARS
8 18 69



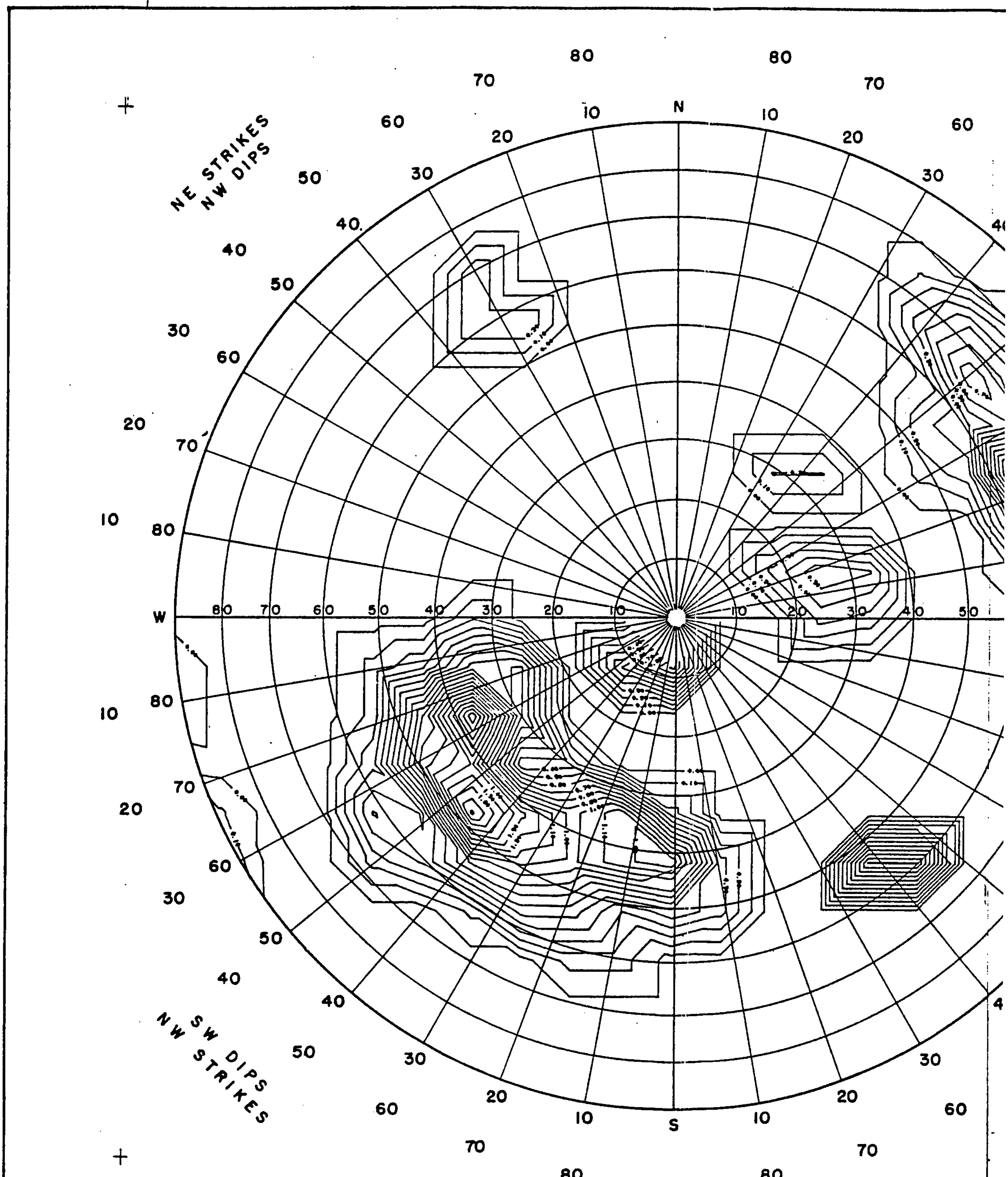
NOTE

Use the outer circle of numbers for reading strike azimuth, the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

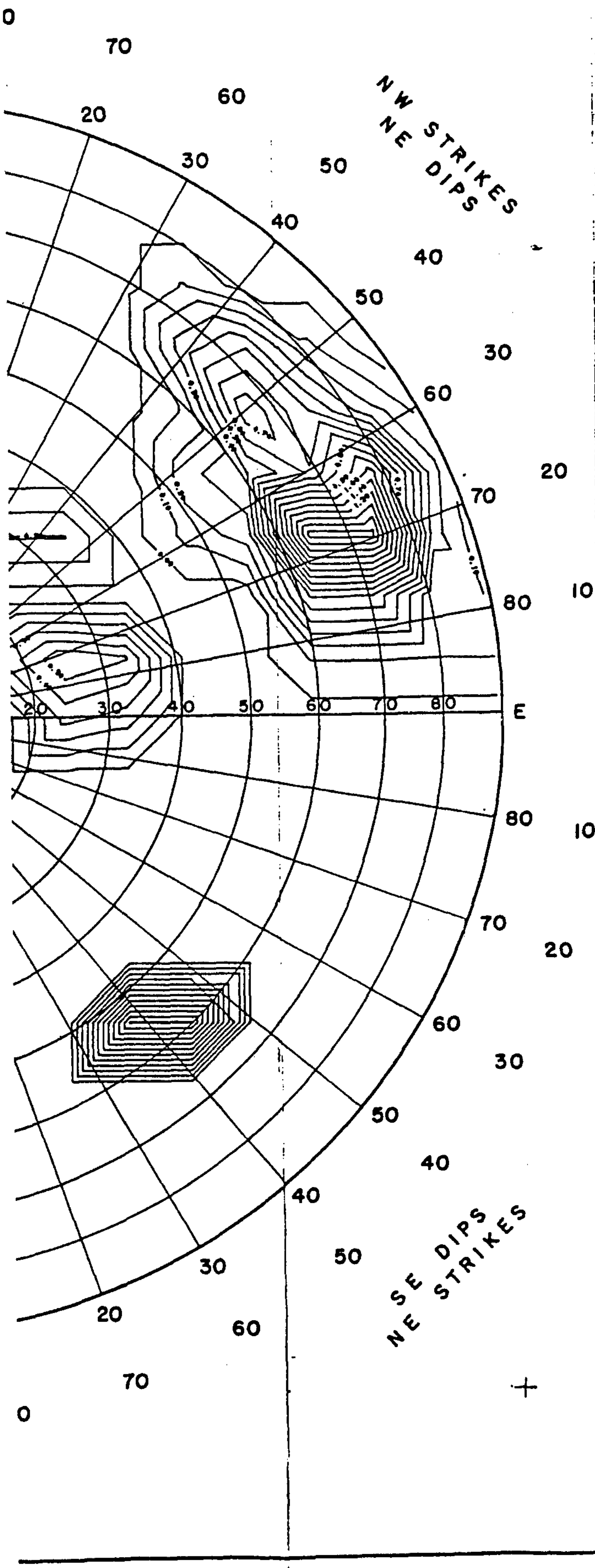
DRAWN ____________ SUBMITTED ____________
TRACED A.P.R. RECOMMENDED ____________
CHECKED ____________ APPROVED ____________

2

6 of 10



TUNNEL 2, DRIFTS 2A, 2B, 2C, BLOCK I, SHEARS
07/04/69



EXPLANATION

Countoured Shear Density Diagram
Equal-Area Projection
Upper Hemisphere

Measurements of true strike and dip of shears are made along a linear traverse, and are plotted using Terzaghi 1/ corrections and hole-length factors. Each shear plane is plotted at the equal-area projection of the intersection of its normal with a reference (upper) hemisphere. The value of the point for contouring purposes equals 100.0 divided by the sine of the angle of intersection of the plane and the traverse, divided by the length of the traverse along which measurements are made. Planes which intersect the traverse at an angle of less than 5° (i.e. within a blind zone of 5°) are not computed.

Points are contoured by summing the value of all points within circles of 1% area on the hemisphere, centered on points which form a grid with a spacing 1/10 of the radius of the reference sphere. No values are computed for grid points which fall within the blind zone.

Contours represent shear frequencies in occurrences per 100 feet normal to each set of shears per 1% area. Contour densities are indicated in the boxes below.

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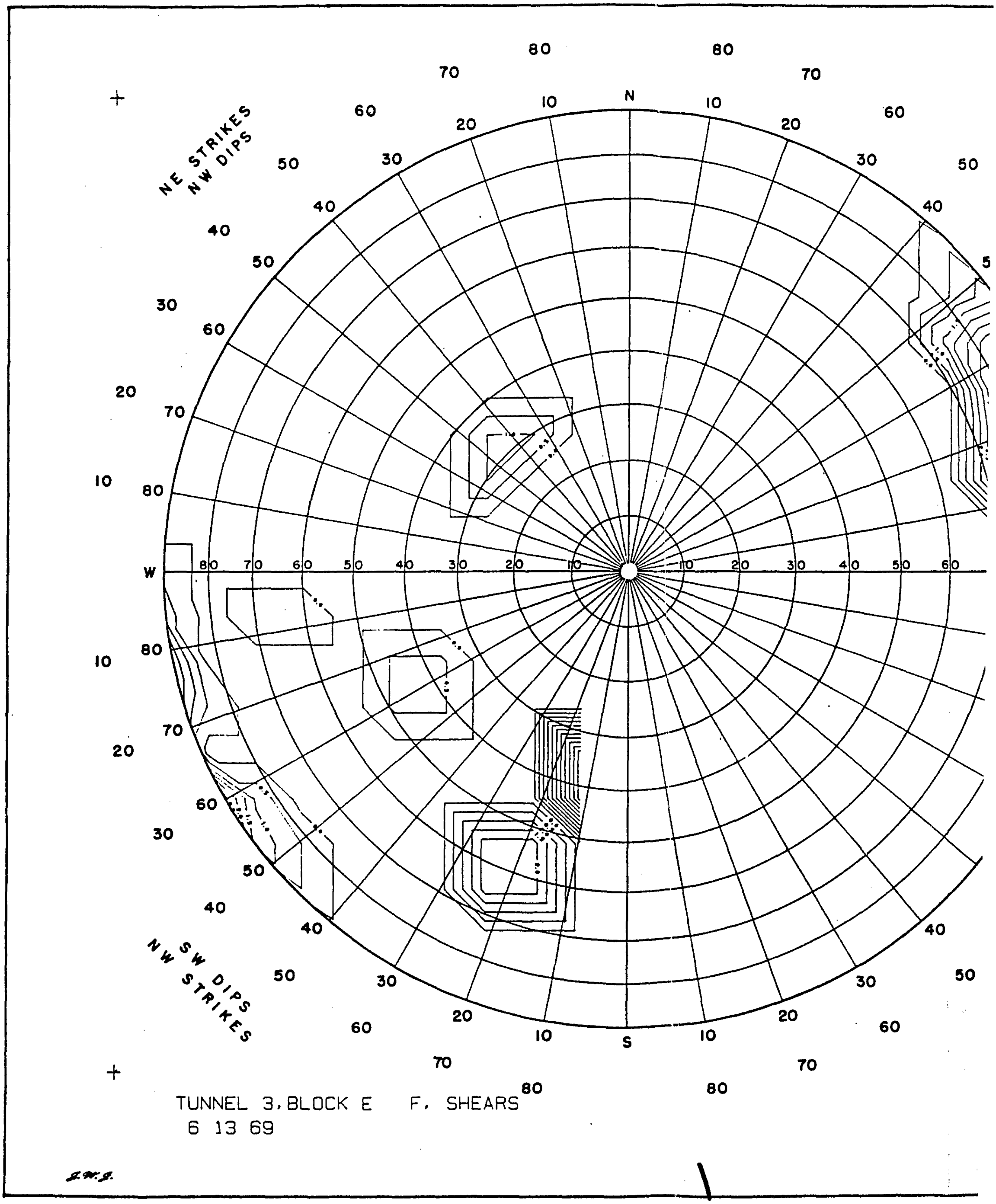
NOTE

Use the outer circle of numbers for reading strike azimuth the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

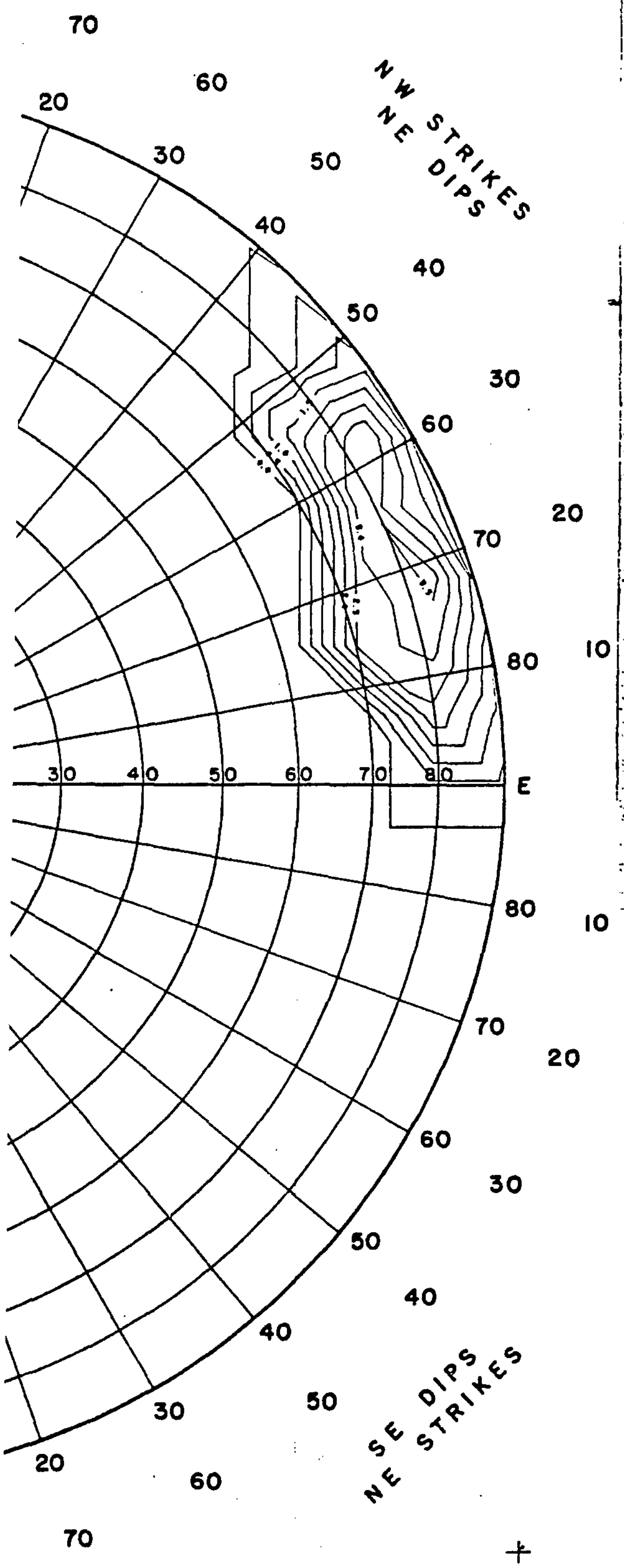
DRAWN ____________ SUBMITTED ____________
TRACED A.P.B. RECOMMENDED ____________
CHECKED ____________ APPROVED ____________

2

70+10



TUNNEL 3, BLOCK E F. SHEARS
6 13 69



EXPLANATION

Countoured Shear Density Diagram
Equal-Area Projection
Upper Hemisphere

Measurements of true strike and dip of shears are made along a linear traverse, and are plotted using Terzaghi 1/ corrections and hole-length factors. Each shear plane is plotted at the equal-area projection of the intersection of its normal with a reference (upper) hemisphere. The value of the point for contouring purposes equals 100.0 divided by the sine of the angle of intersection of the plane and the traverse, divided by the length of the traverse along which measurements are made. Planes which intersect the traverse at an angle of less than 5 ° (i.e. within a blind zone of 5 °) are not computed.

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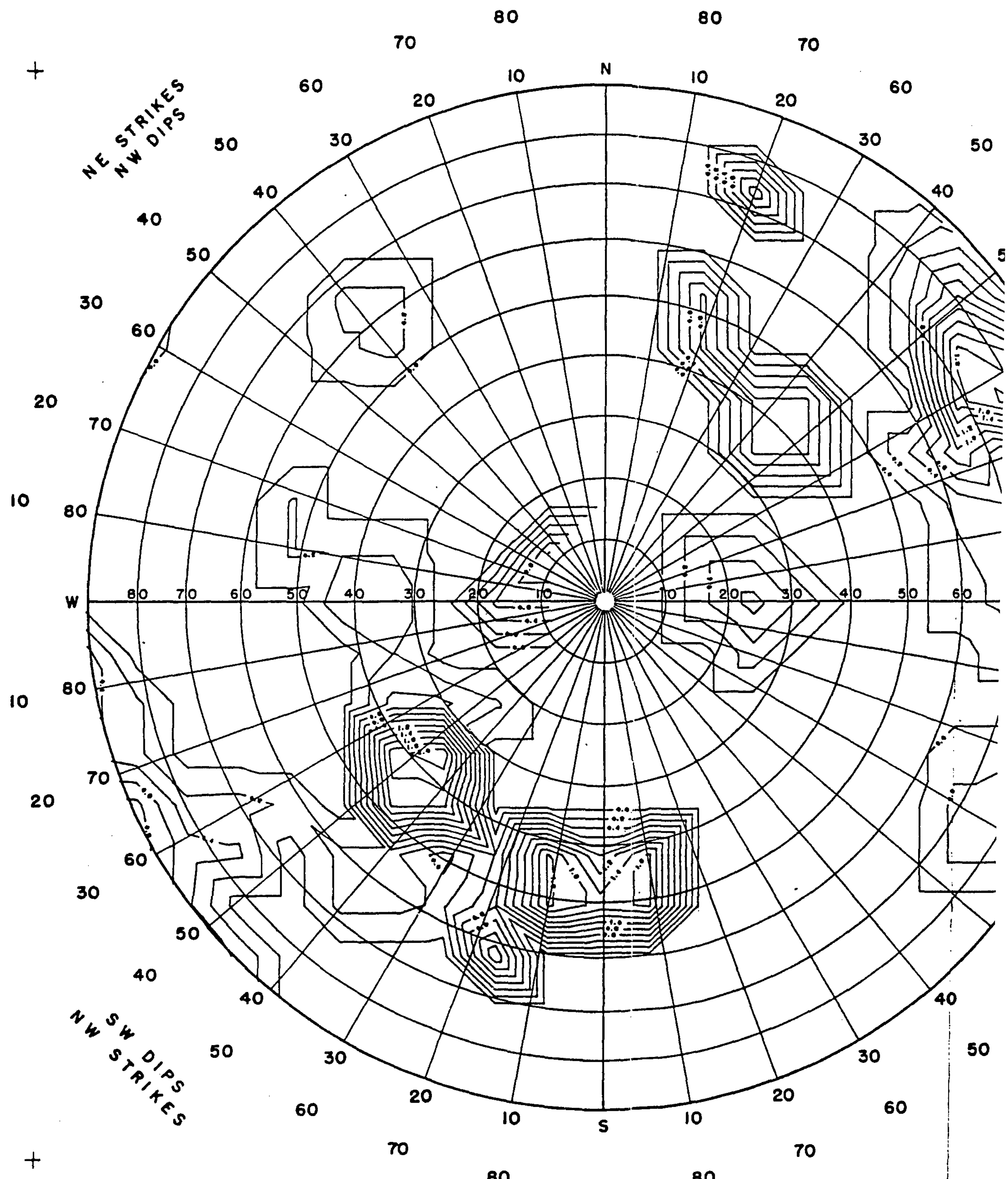
DRAWN ____ SUBMITTED ____
TRACED A.P.R. RECOMMENDED ____
CHECKED ____ APPROVED ____

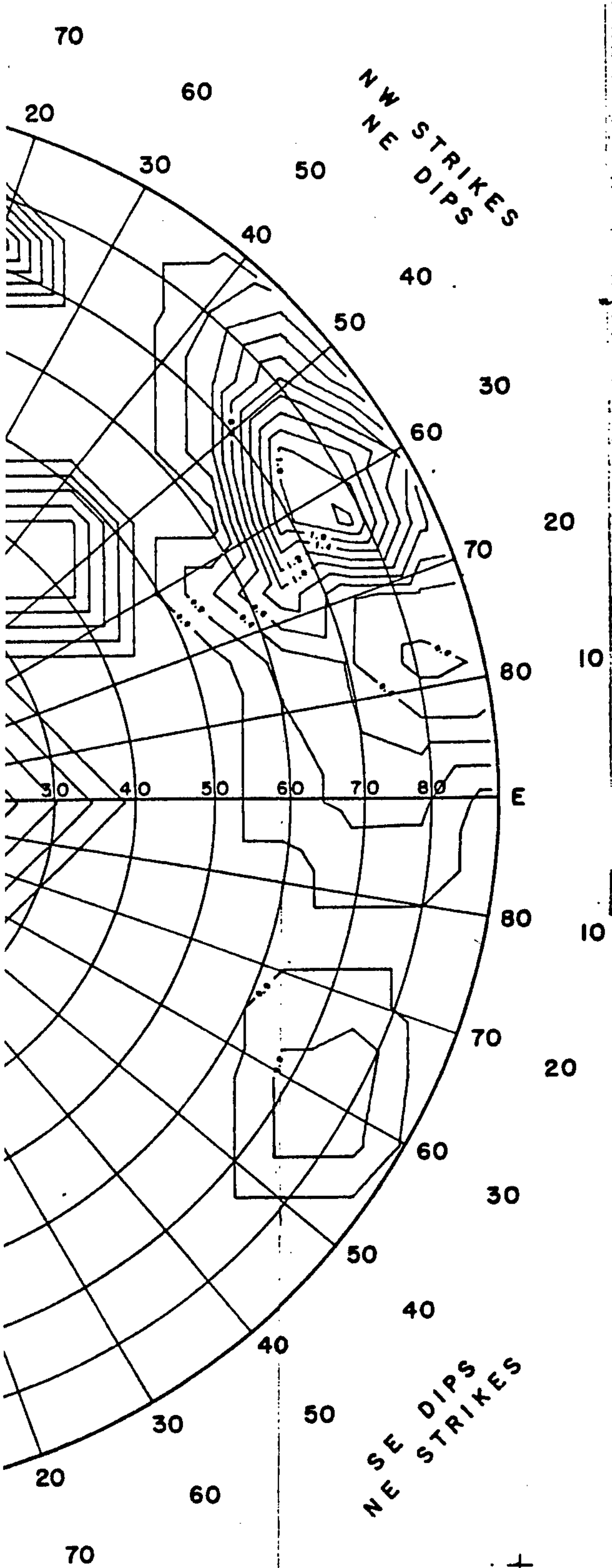
2

8 of 10



TUNNEL 4. DRFTS 4A. 4B. BLOCKS C. D. E. SHEARS
07/04/69

J.W.J.



EXPLANATION

Countoured Shear Density Diagram
Equal-Area Projection
Upper Hemisphere

Measurements of true strike and dip of shears are made along a linear traverse, and are plotted using Terzaghi 1/ corrections and hole-length factors. Each shear plane is plotted at the equal-area projection of the intersection of its normal with a reference (upper) hemisphere. The value of the point for contouring purposes equals 100.0 divided by the sine of the angle of intersection of the plane and the traverse, divided by the length of the traverse along which measurements are made. Planes which intersect the traverse at an angle of less than 5 ° (i.e. within a blind zone of 5 °) are not computed.

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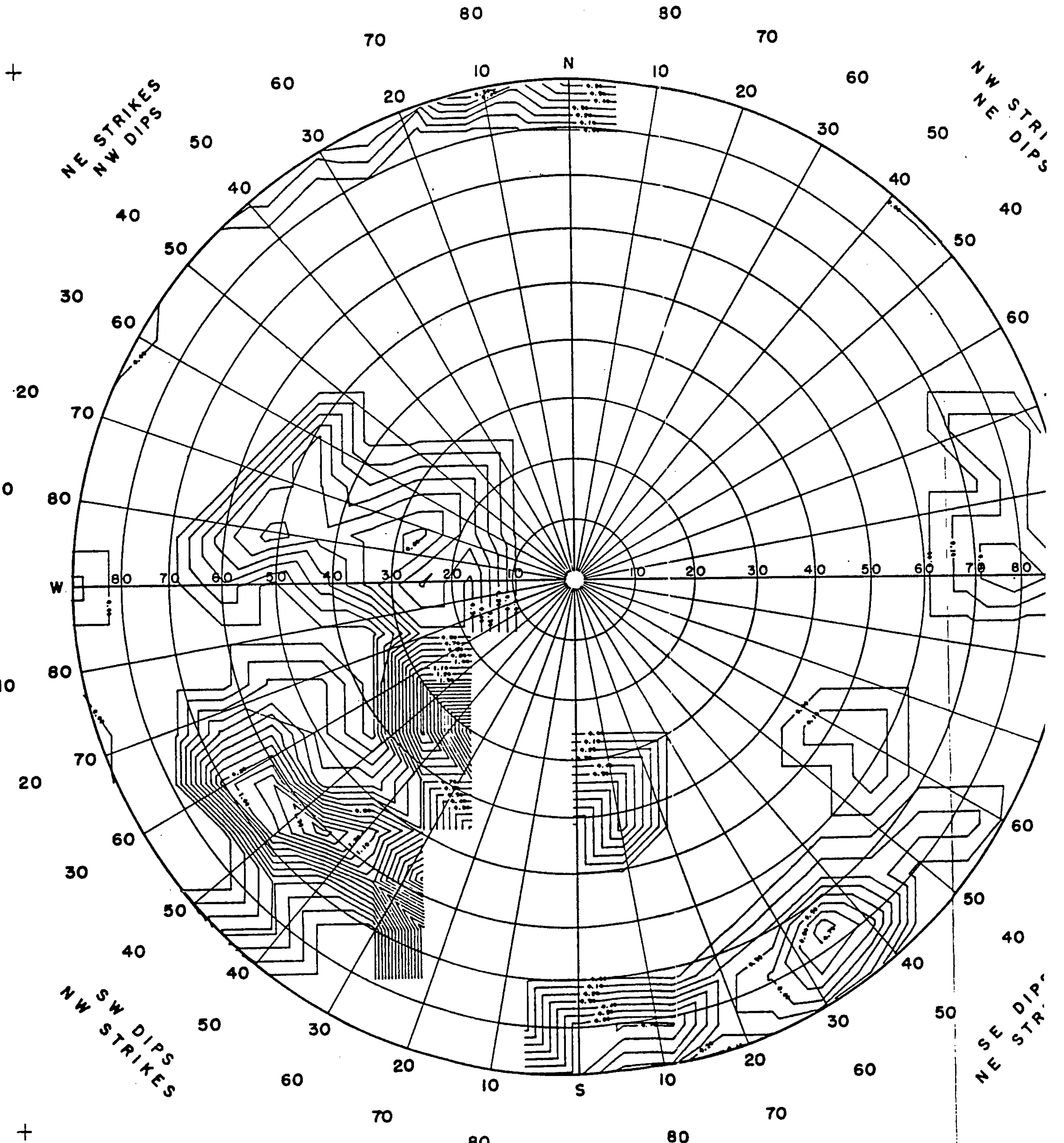
NOTE

Use the outer circle of numbers for reading strike azimuth the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

DRAWN	SUBMITTED
TRACED A.P.R.	RECOMMENDED
CHECKED	APPROVED

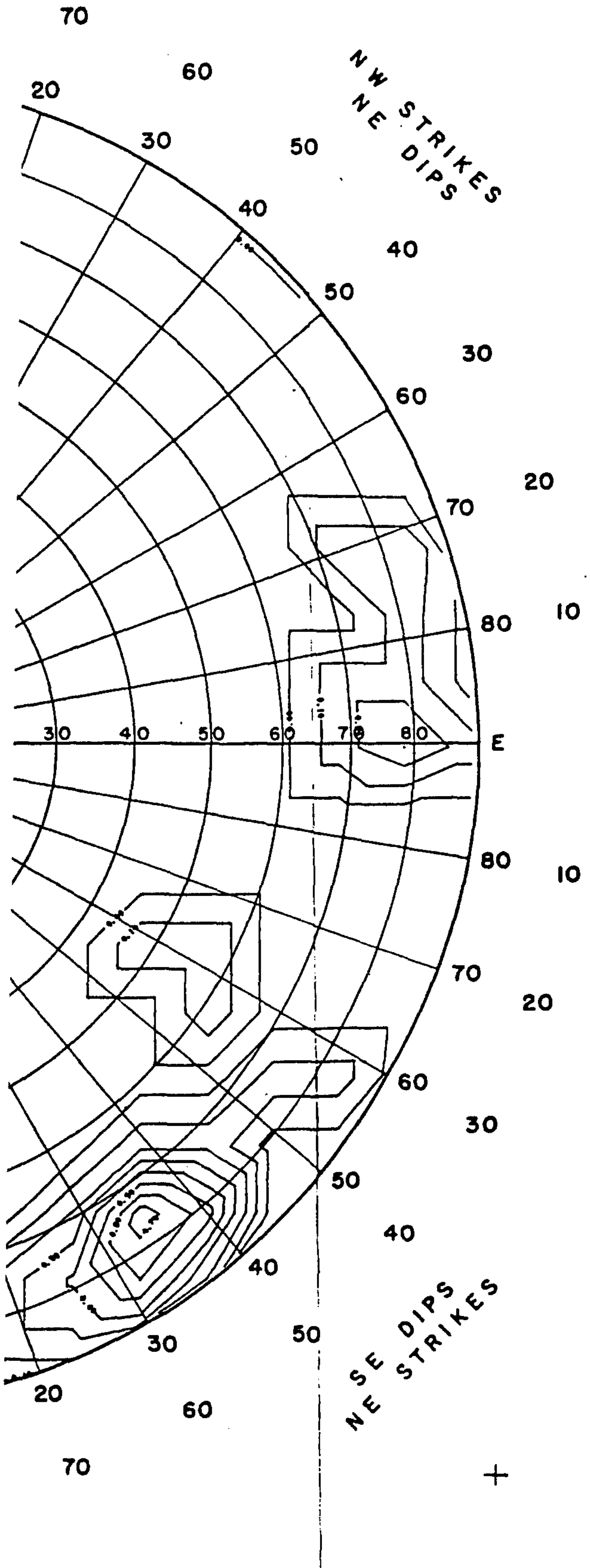
2

96410



TUNNEL 5, BLOCK C, SHEARS
6 13 69

J.W.J.



EXPLANATION

Countoured Shear Density Diagram
Equal-Area Projection
Upper Hemisphere

Measurements of true strike and dip of shears are made along a linear traverse, and are plotted using Terzaghi 1/ corrections and hole-length factors. Each shear plane is plotted at the equal-area projection of the intersection of its normal with a reference (upper) hemisphere. The value of the point for contouring purposes equals 100.0 divided by the sine of the angle of intersection of the plane and the traverse, divided by the length of the traverse along which measurements are made. Planes which intersect the traverse at an angle of less than 5 ° (i.e. within a blind zone of 5 °) are not computed.

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NOTE

Use the outer circle of numbers for reading strike azimuth, the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

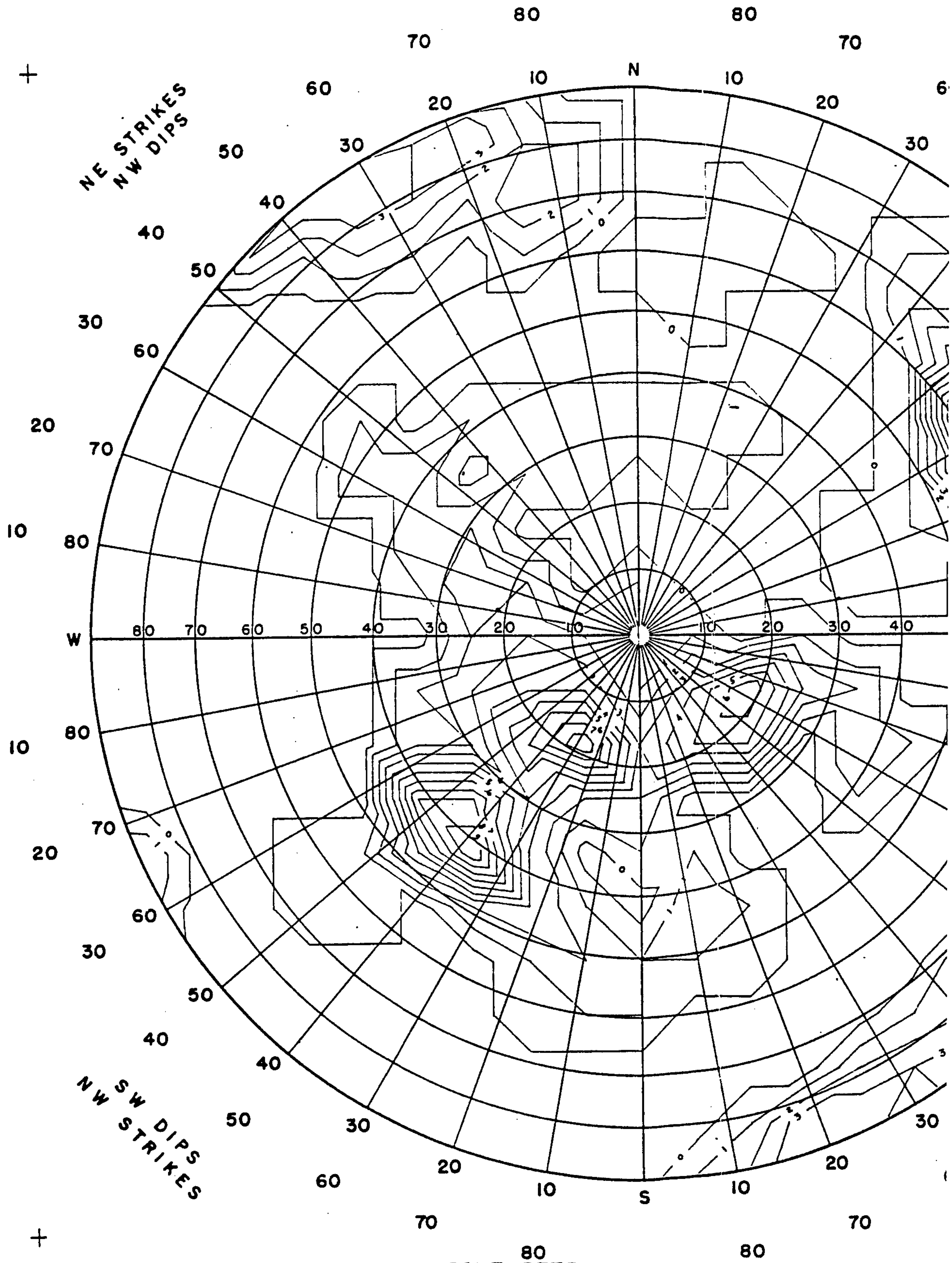
DRAWN .... SUBMITTED ....
TRACED ... A.R.S ... RECOMMENDED ....
CHECKED .... APPROVED ....

2

100of10

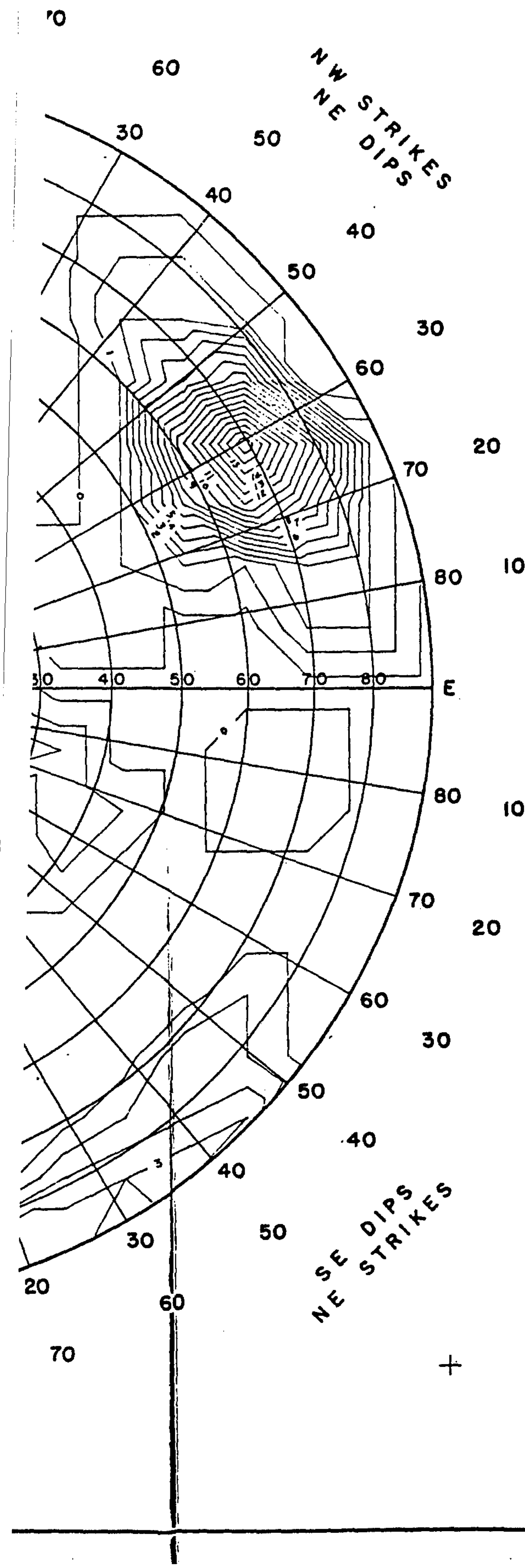
APPENDIX N-3-B

CONTOURED JOINT DIAGRAMS



TUNNEL 2, DR2A, 2B, 2C, JOINT SETS
3 22 69

J.W.J.



EXPLANATION

Contoured Joint Diagram
Equal-Area Projection
Upper Hemisphere

Each joint set as recorded in logs of tunnels, drifts, and/or raises is plotted at the equal-area projection of the intersection of its normal with a reference (upper) hemisphere. The value of the point for contouring purposes is 1.0.

Points are contoured by summing the value of all points within circles of 1% area on the hemisphere, centered on points which form a grid with a spacing 1/10 of the radius of the reference sphere.

Contours represent number of joint sets per 1% area. Contour densities indicate the orientation and approximate relative frequency of occurrence of important joint sets. No information on the spacing of the joints within the various sets can be obtained from this drawing.

Contour densities are indicated in the boxes below.

1 FOOT CONTOURS

0-1	1-2	2-3	3-4	4-5
5-6	6-7	7-8	8-9	9-10
10-11	11-12	12-13	13-14	14-15

NOTE

Use the outer circle of numbers for reading strike azimuth, the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT
AUBURN-FOLSOM SOUTH UNIT-CALIF.

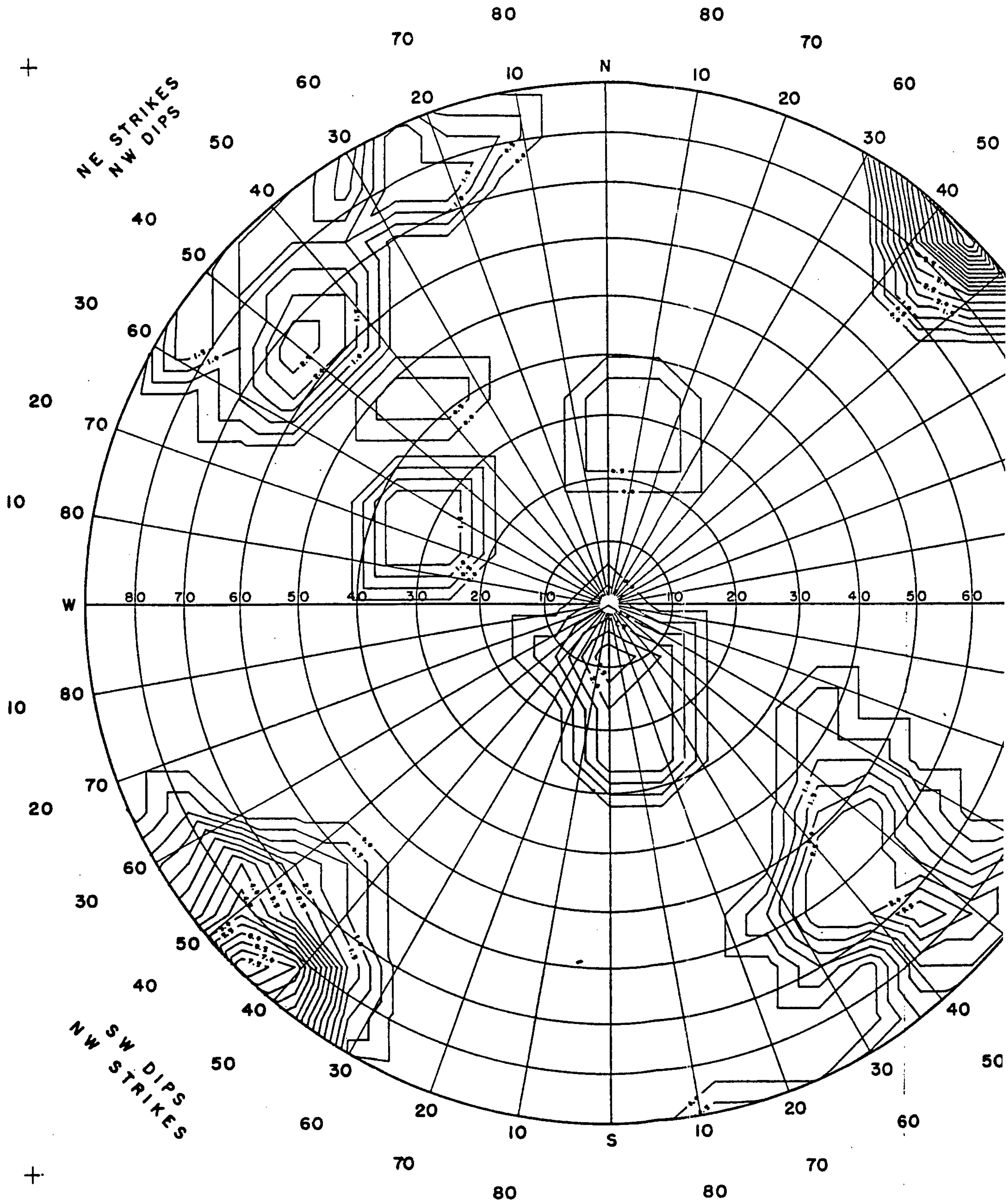
AUBURN DAM
TUNNEL 2, DRIFT 2A, 2B, 2C
JOINT SETS

DRAWN............SUBMITTED............
TRACED....A.R.B....RECOMMENDED............
CHECKED............APPROVED............

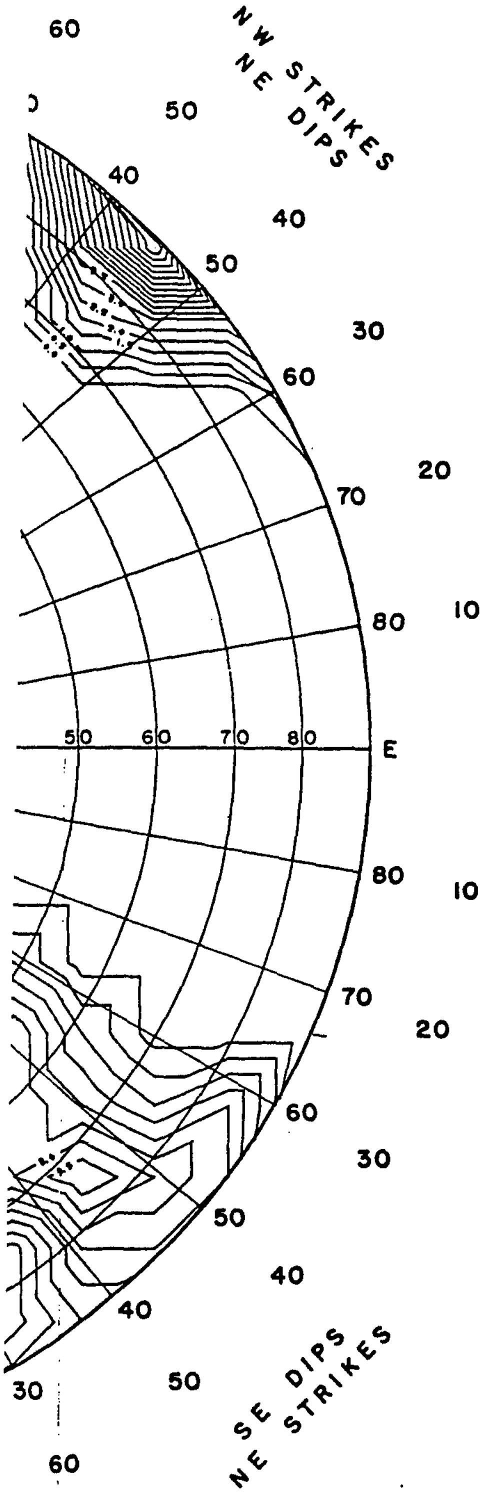
DENVER, COLORADO, | 859-D-174

2

1of16



TU 1, DR1A, 1B, BLOCK J, JOINT SETS
6 20 69

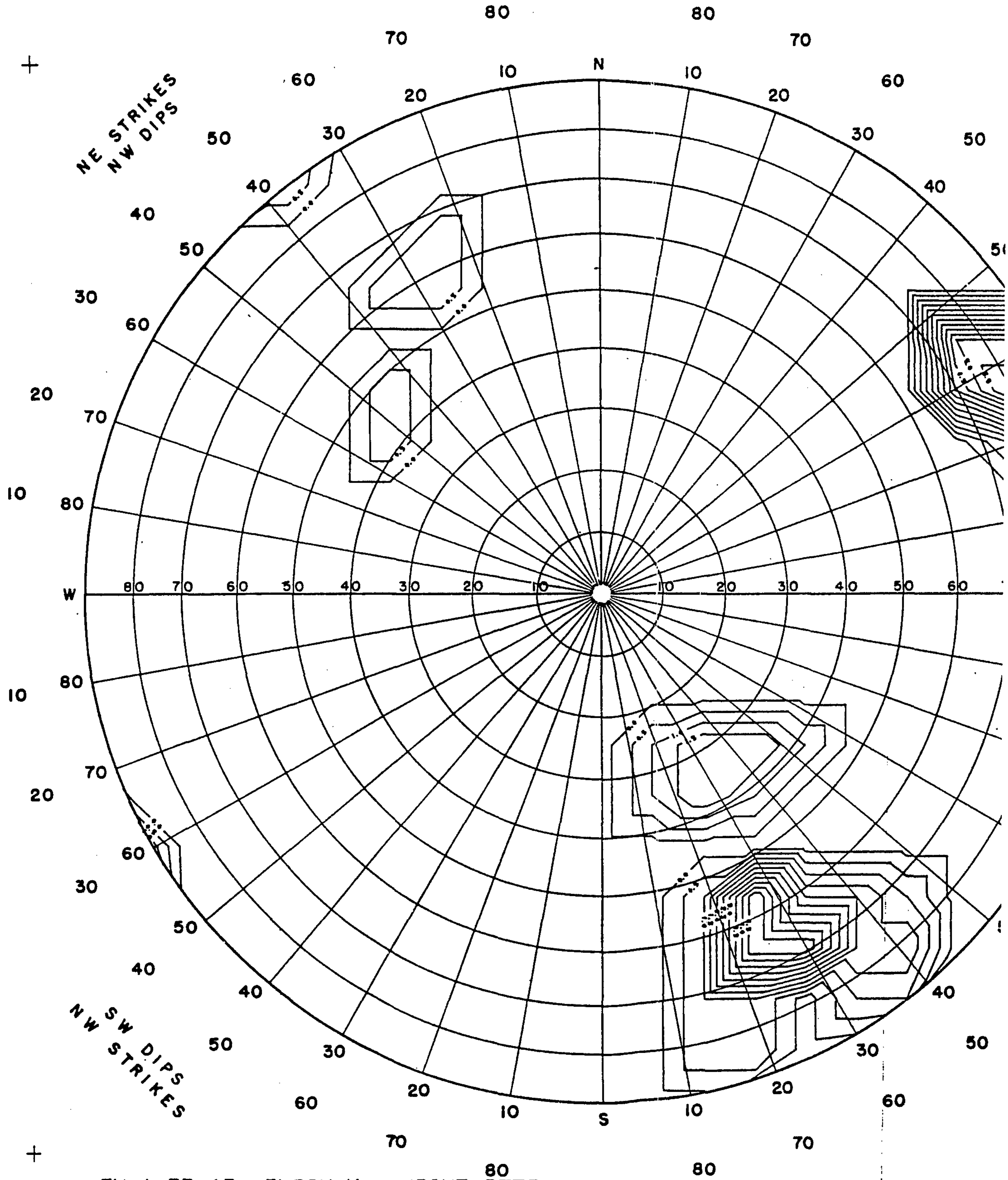


NOTE

Use the outer circle of numbers for reading strike azimuth, the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

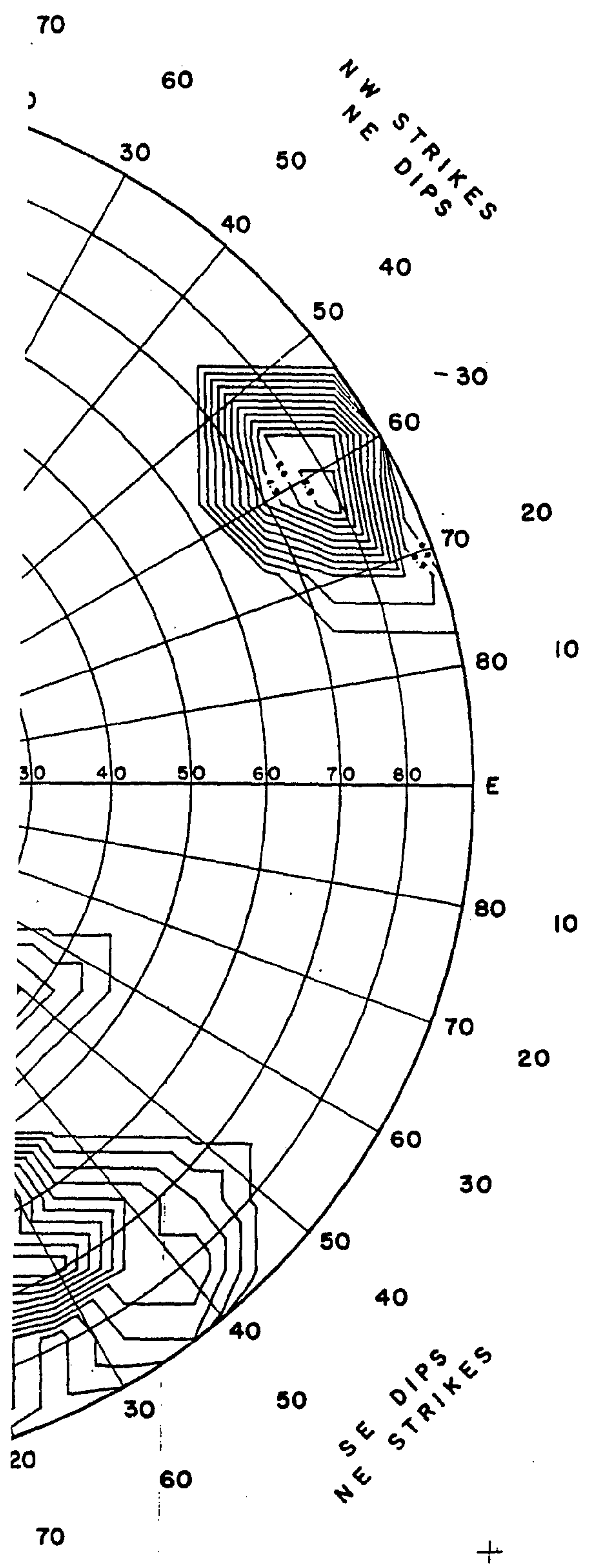
DRAWN ______ SUBMITTED ______
TRACED A.P.R. RECOMMENDED ______
CHECKED ______ APPROVED ______

2



TU 1, DR 1B, BLOCK K , JOINT SETS
6 20 69

J.W.J.



NOTE

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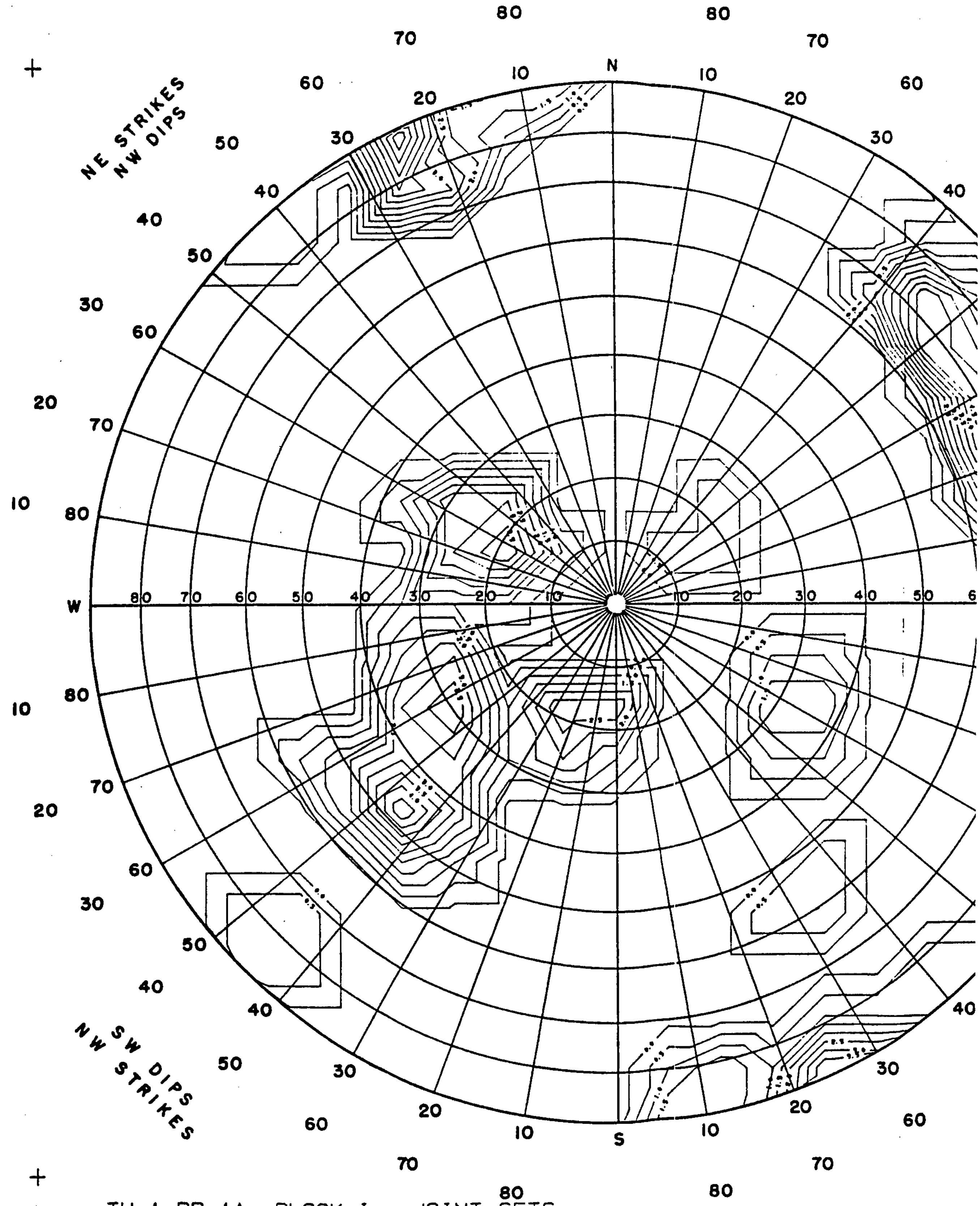
DRAWN .... SUBMITTED ....
TRACED .... A.R.R. .... RECOMMENDED ....
CHECKED .... APPROVED ....

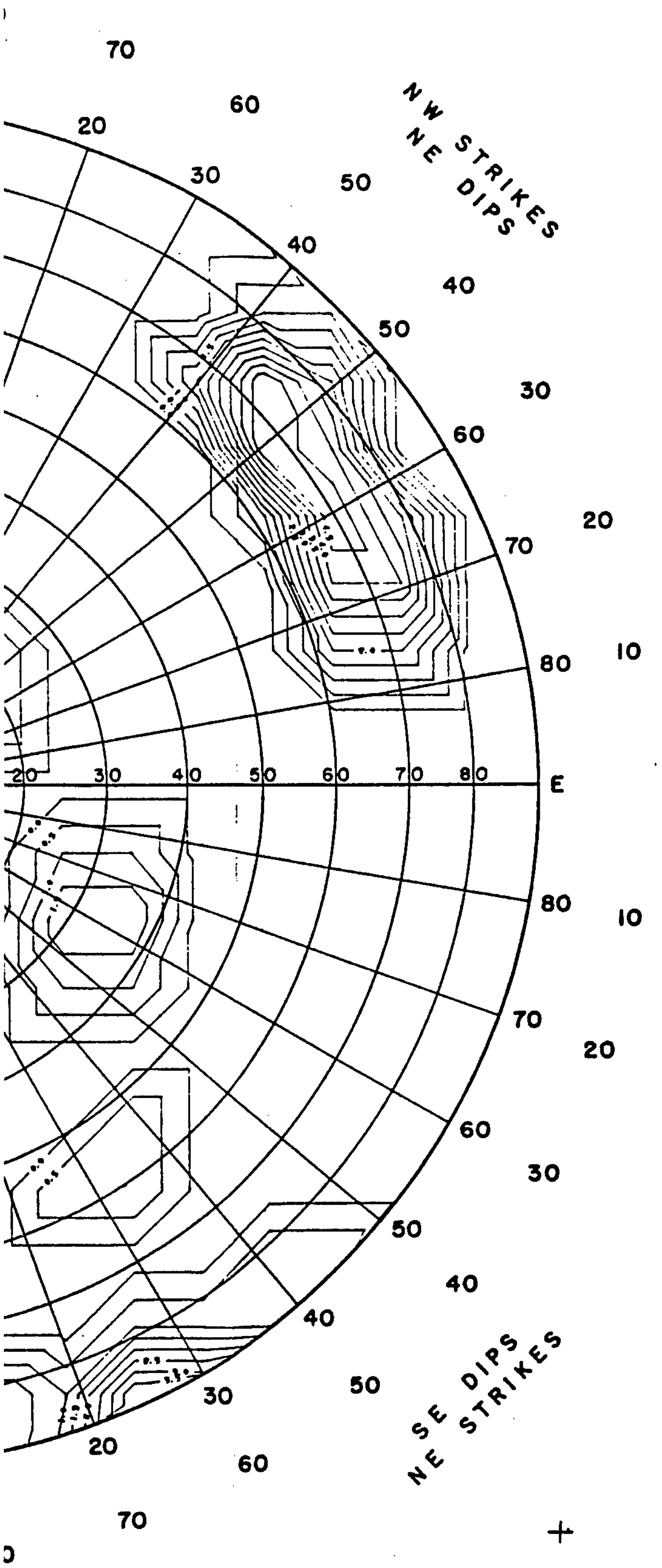
2



TU 1, DR 1A, BLOCK I, JOINT SETS
6 20 69

J.W.J.

1

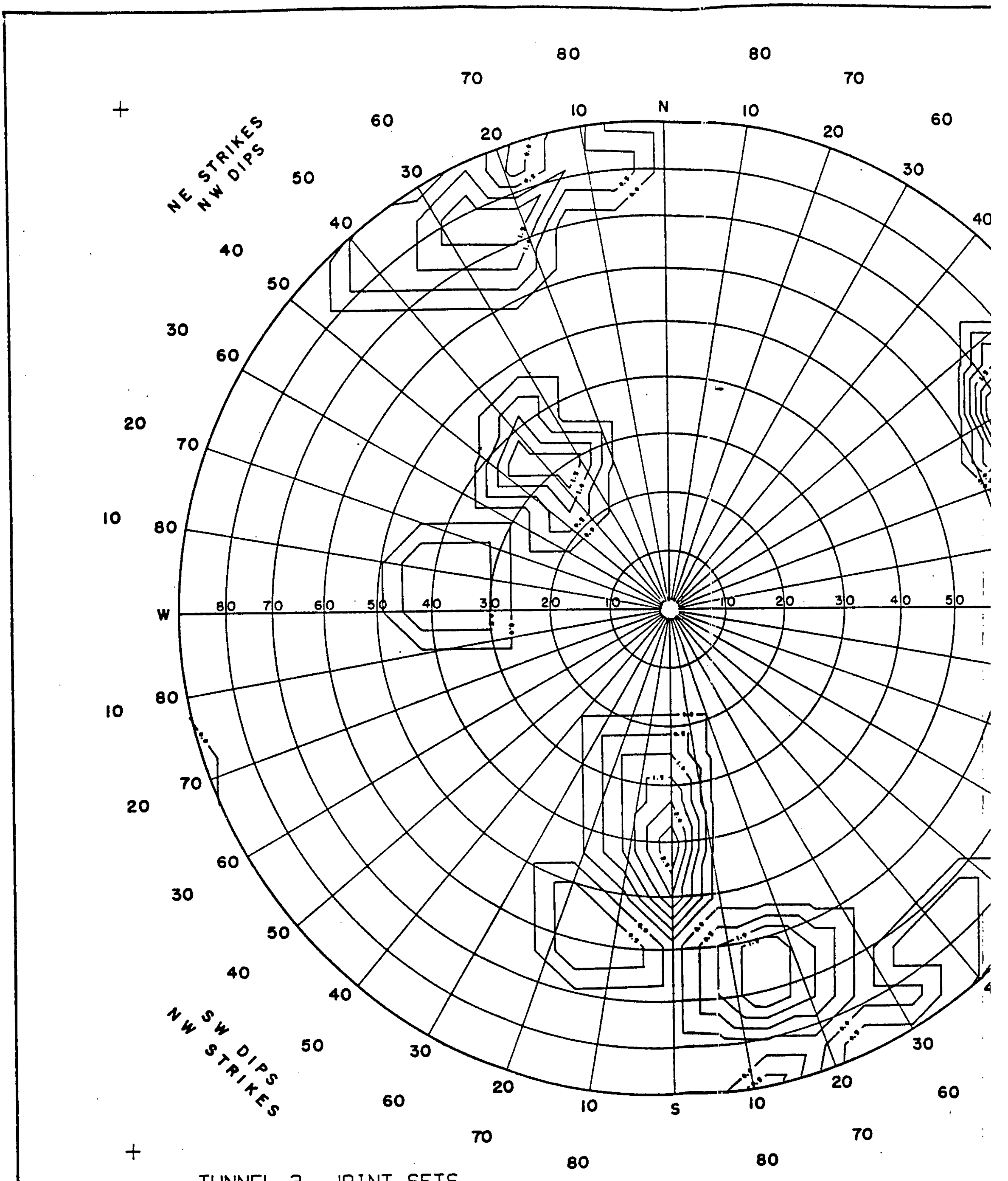


NOTE

Use the outer circle of numbers for reading strike azimuth, the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

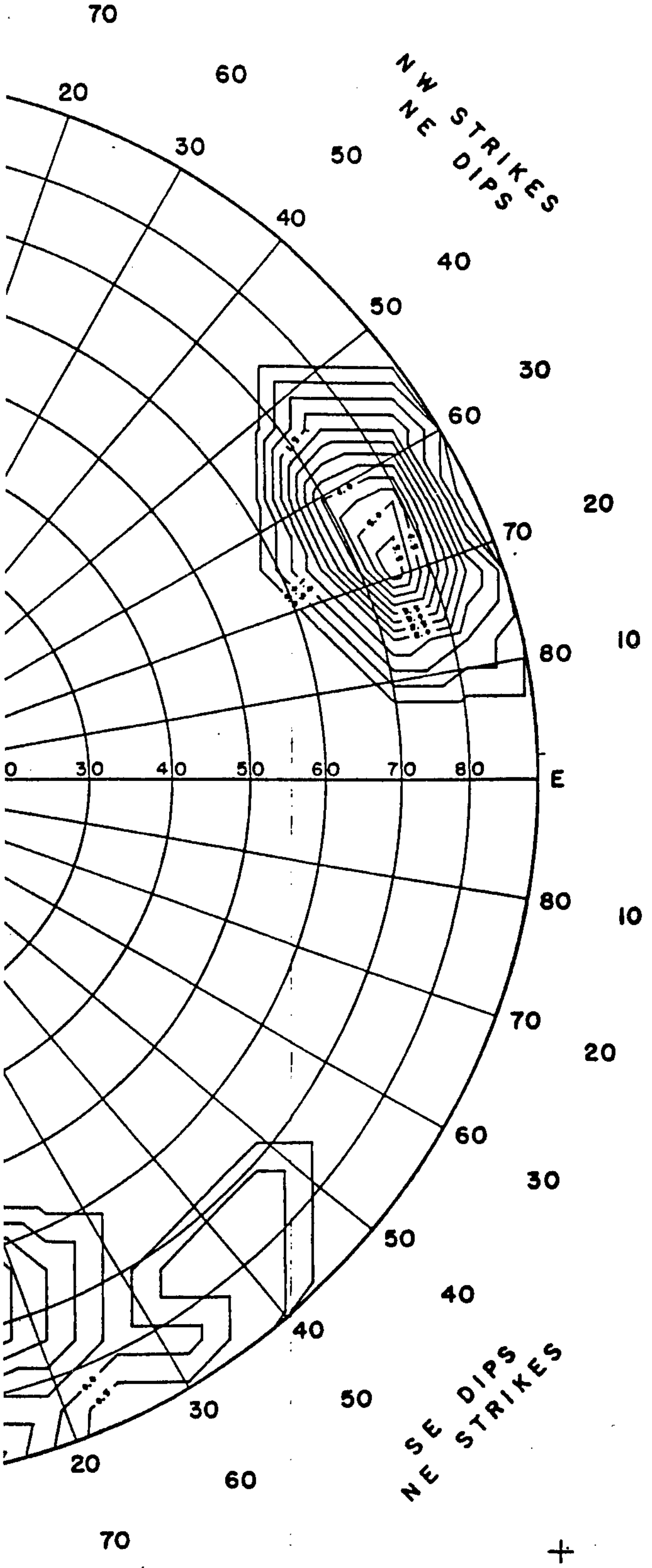
DRAWN ______ SUBMITTED ______
TRACED ___A.R.S.___ RECOMMENDED ______
CHECKED ______ APPROVED ______

2



TUNNEL 3, JOINT SETS
6 20 69

J.W.J.

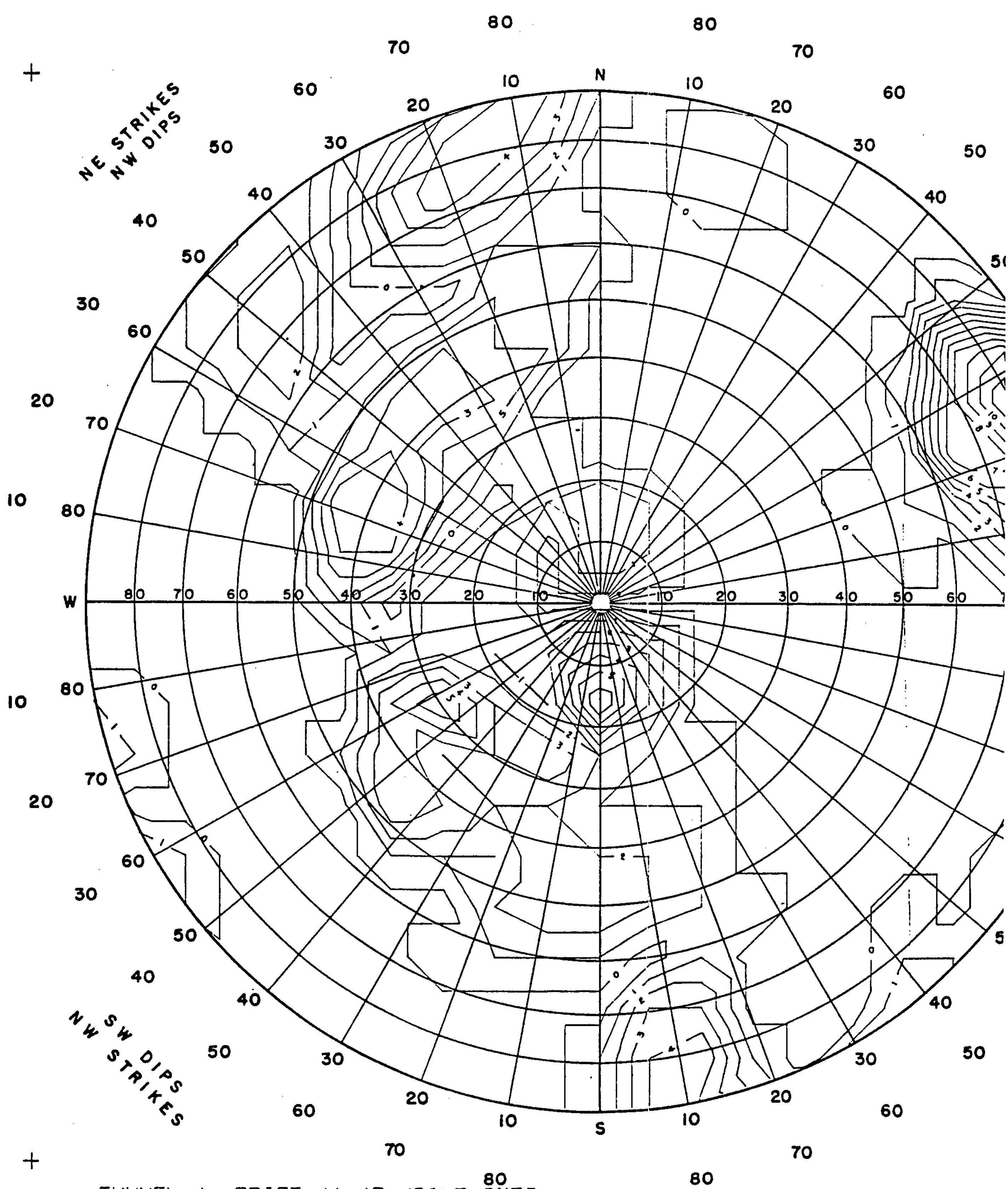


NOTE

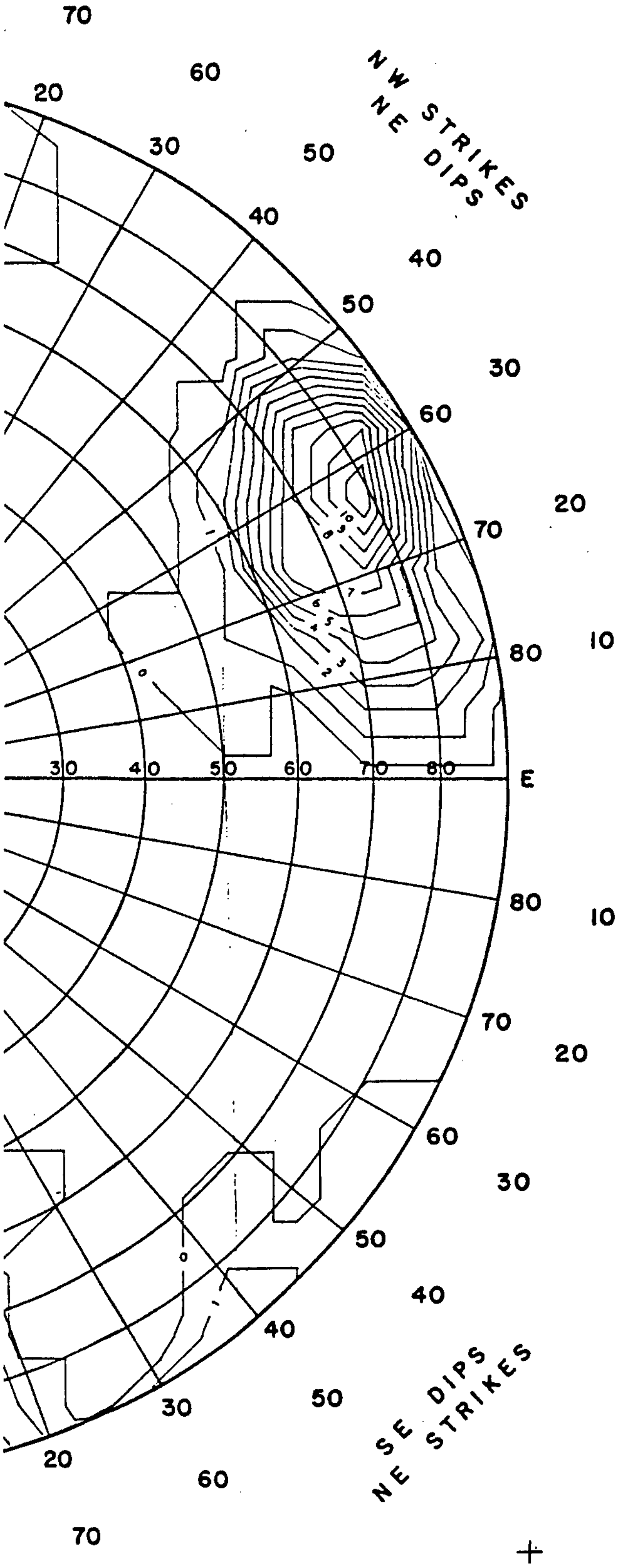
Use the outer circle of numbers for reading strike azimuth, the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

2

5 of 16



TUNNEL 4, DRIFT 4A,4B,JOINT SETS
3 22 69



EXPLANATION

Contoured Joint Diagram
Equal-Area Projection
Upper Hemisphere

Each joint set as recorded in logs of tunnels, drifts, and/or raises is plotted at the equal-area projection of the intersection of its normal with a reference (upper) hemisphere. The value of the point for contouring purposes is 1.0.

Points are contoured by summing the value of all points within circles of 1% area on the hemisphere, centered on points which form a grid with a spacing 1/10 of the radius of the reference sphere.

Contours represent number of joint sets per 1% area. Contour densities indicate the orientation and approximate relative frequency of occurrence of important joint sets. No information on the spacing of the joints within the various sets can be obtained from this drawing.

Contour densities are indicated in the boxes below.

1 FOOT CONTOURS

0-1	1-2	2-3	3-4	4-5
5-6	6-7	7-8	8-9	9-10

NOTE

Use the outer circle of numbers for reading strike azimuth, the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT
AUBURN-FOLSOM SOUTH UNIT-CALIF.

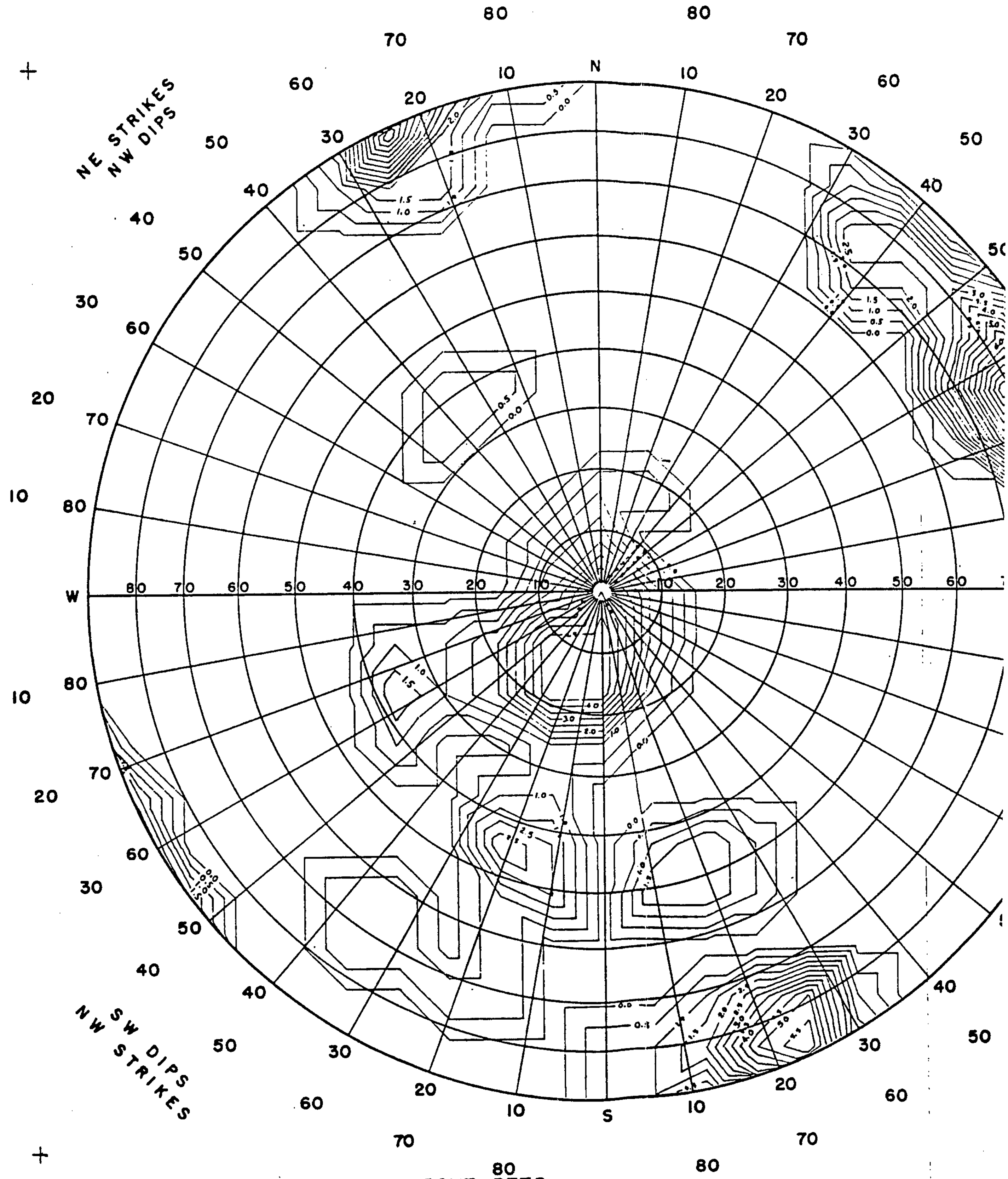
AUBURN DAM
TUNNEL 4, DRIFT 4A, 4B
JOINT SETS

DRAWN.......... SUBMITTED..........
TRACED... A.R.B. ... RECOMMENDED..........
CHECKED.......... APPROVED..........

2

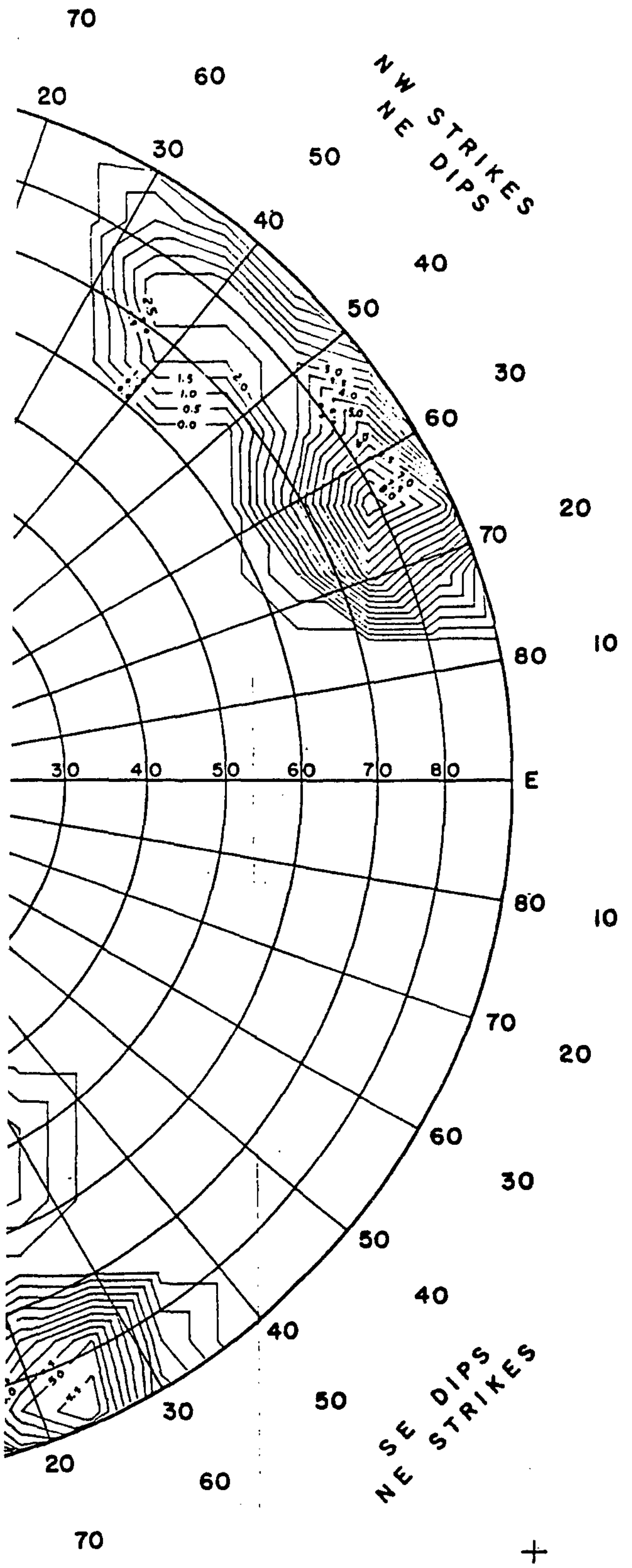
DENVER, COLORADO 859-D-173

6 of 16



TUNNEL 5, STA 257-700, JOINT SETS
3 22 69

J. W. J.



EXPLANATION

Contoured Joint Diagram
Equal-Area Projection
Upper Hemisphere

Each joint set as recorded in logs of tunnels, drifts, and/or raises is plotted at the equal-area projection of the intersection of its normal with a reference (upper) hemisphere. The value of the point for contouring purposes is 1.0.

Points are contoured by summing the value of all points within circles of 1% area on the hemisphere, centered on points which form a grid with a spacing 1/10 of the radius of the reference sphere.

Contours represent number of joint sets per 1% area. Contour densities indicate the orientation and approximate relative frequency of occurrence of important joint sets. No information on the spacing of the joints within the various sets can be obtained from this drawing.

Contour densities are indicated in the boxes below.

0.5 FT. CONTOURS

0.0-0.5	0.5-1.0	1.0-1.5	1.5-2.0	2.0-2.5	2.5-3.0
3.0-3.5	3.5-4.0	4.0-4.5	4.5-5.0	5.0-5.5	5.5-6.0
6.0-6.5	6.5-7.0	7.0-7.5	7.5-8.0		

NOTE

Use the outer circle of numbers for reading strike azimuth, the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

UNITED STATES
DEPARTMENT OF THE INTERIOR
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CENTRAL VALLEY PROJECT
AUBURN-FOLSOM SOUTH UNIT-CALIF.

AUBURN DAM

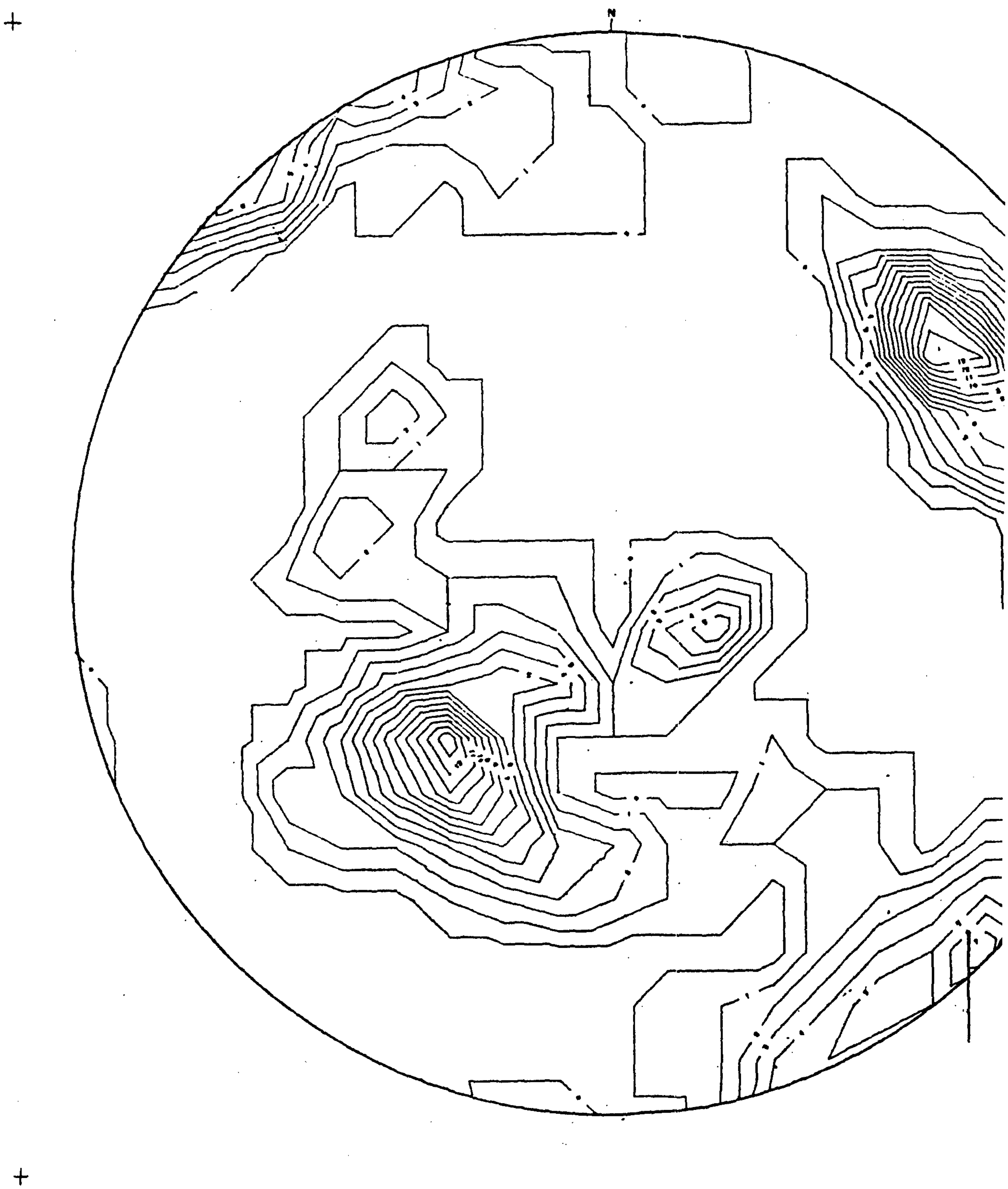
TUNNEL 5, STA. 257-700
JOINT SETS

DRAWN .... SUBMITTED ....
TRACED .... A.A.S. .... RECOMMENDED ....
CHECKED .... APPROVED ....

DENVER, COLORADO, | 859-D-172

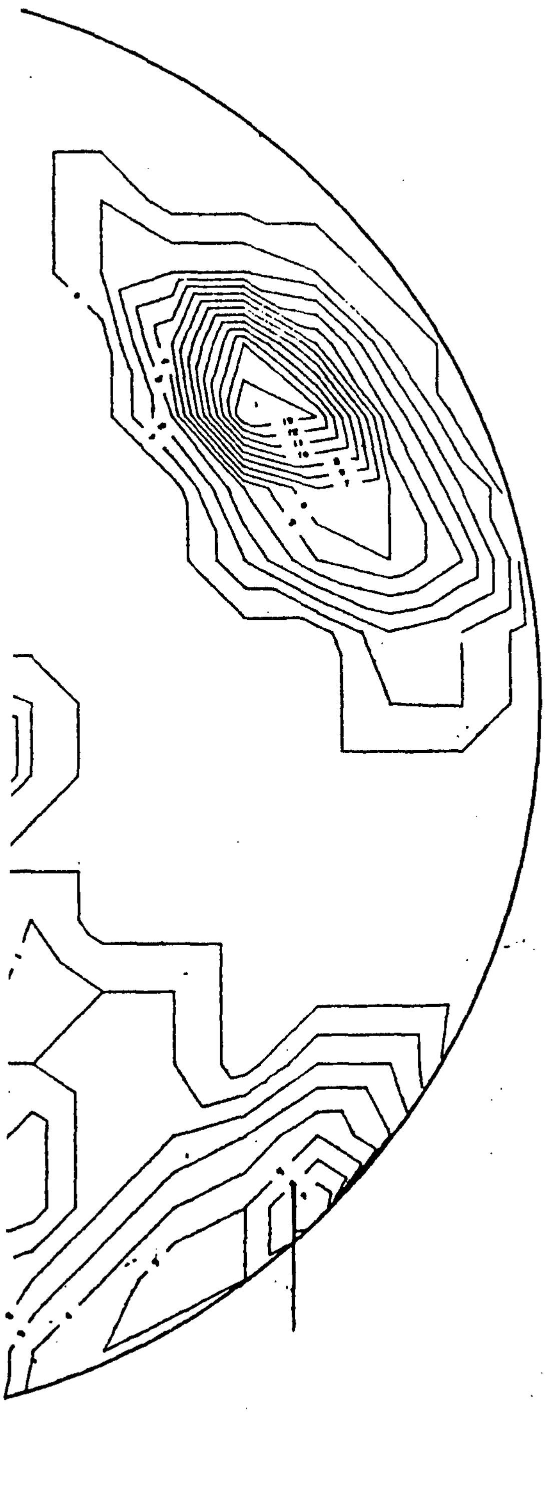
2

7 of 16



AUBURN DAM TUNNEL NO 2
04/09/68

Data from face logs



ALWAYS THINK SAFETY

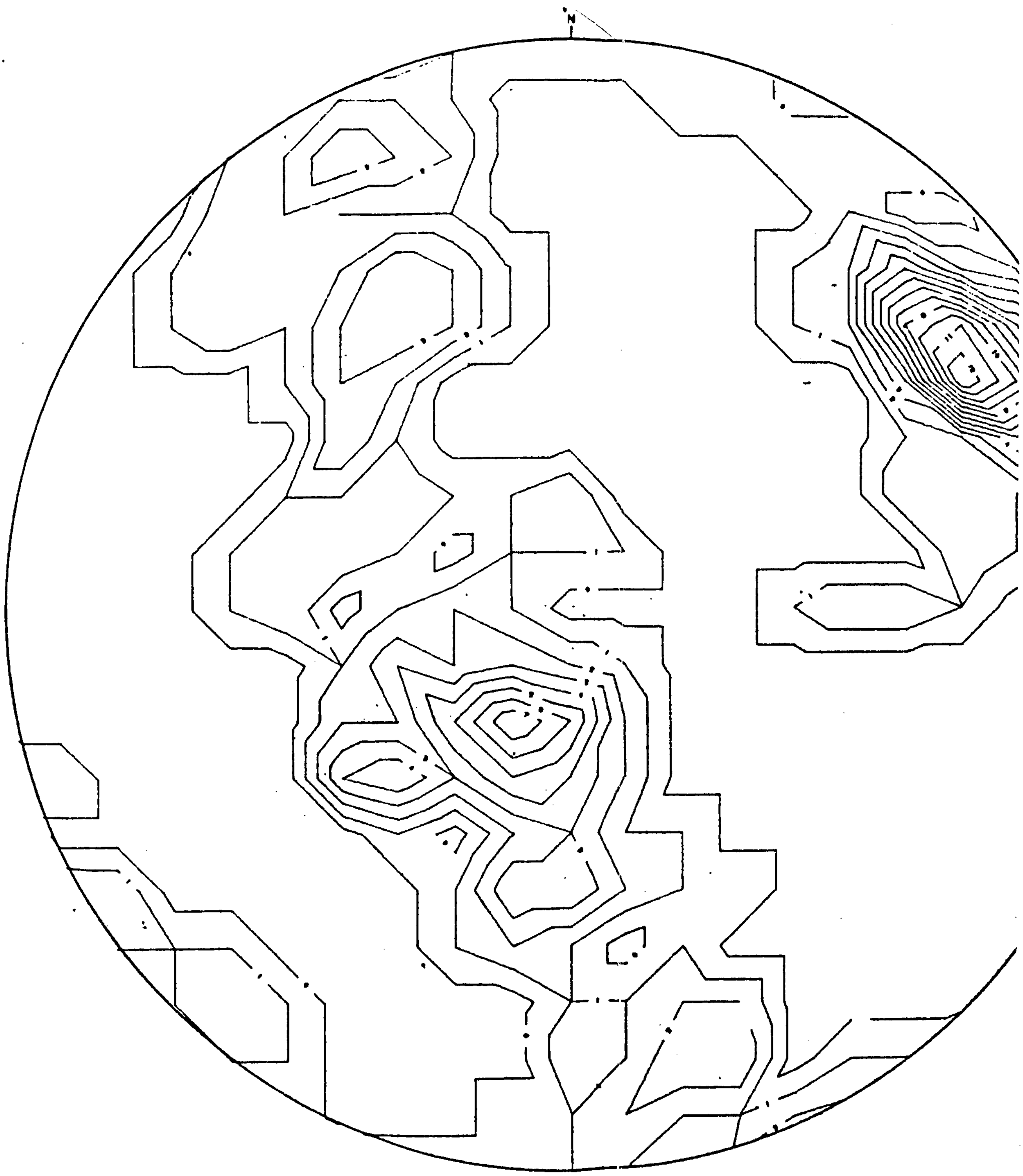
TUNNEL #2

DRAWN ______ SUBMITTED ______
TRACED ______ RECOMMENDED ______
CHECKED ______ APPROVED ______

DENVER, COLORADO

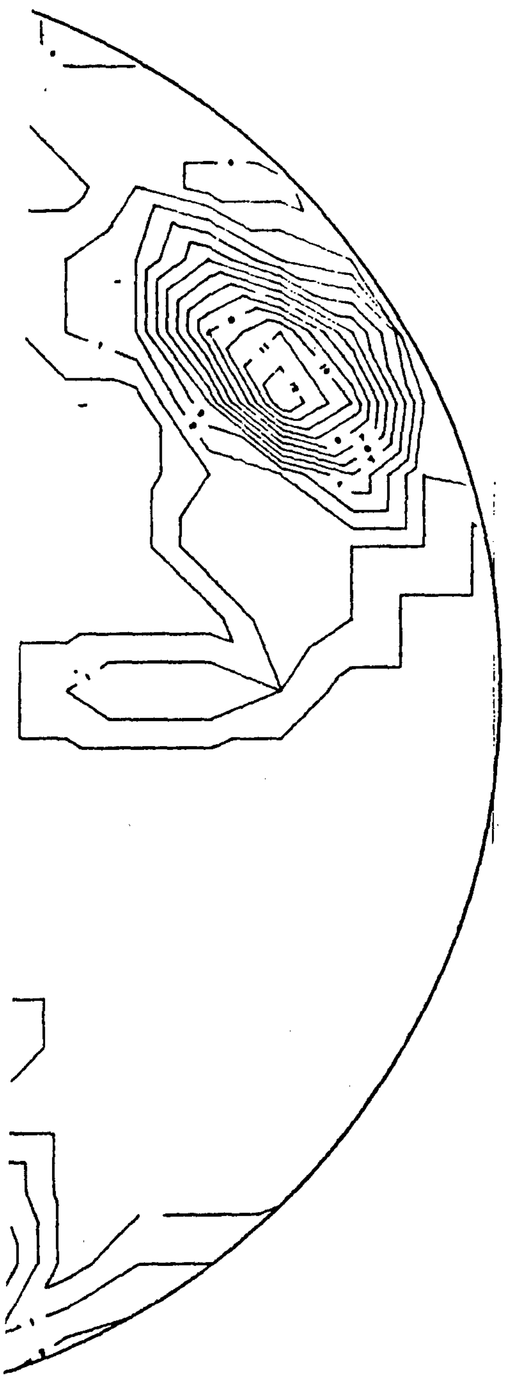
2

8 of 16



AUBURN DAM TUNNEL NO 4
04/09/68

Data from face logs



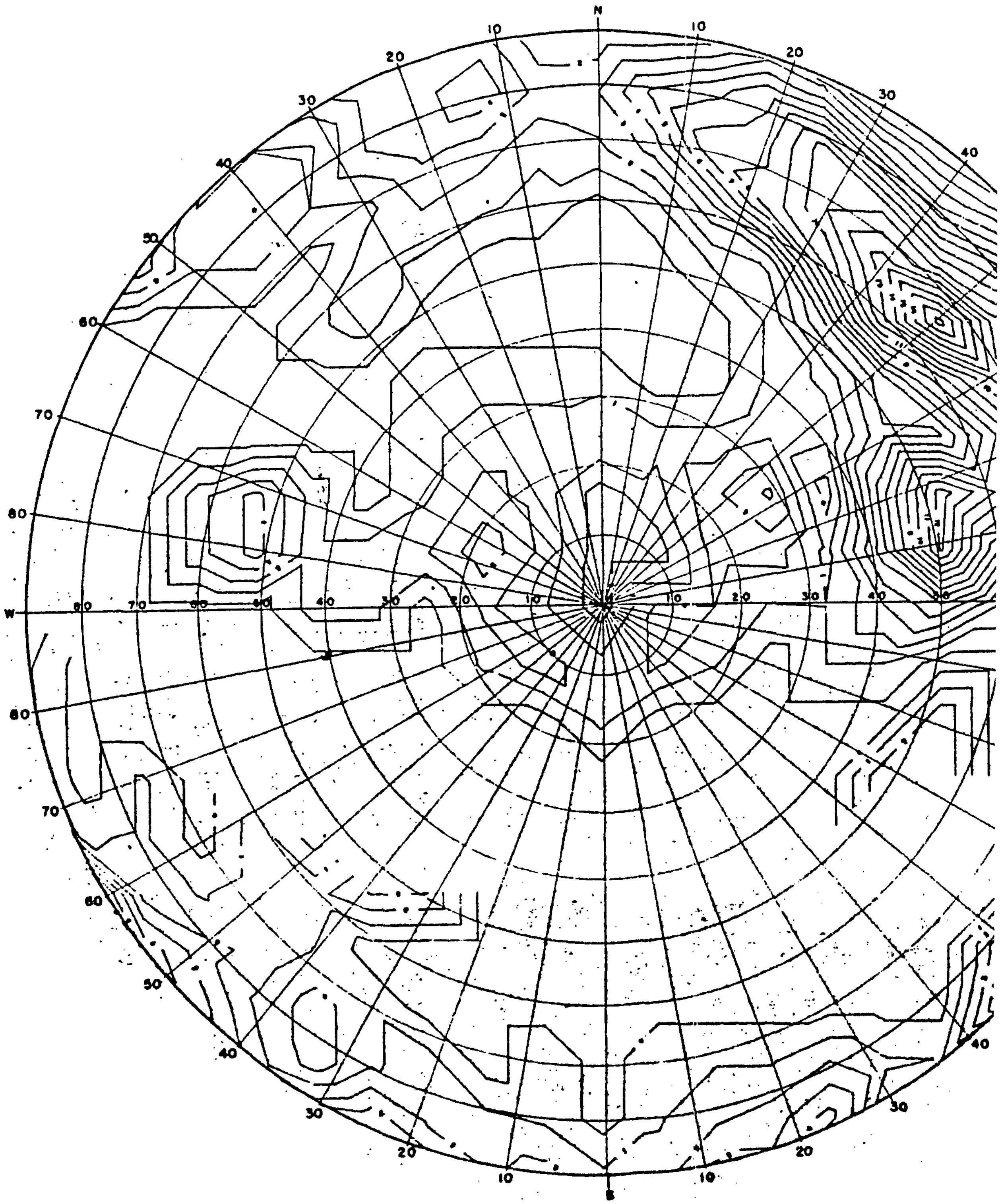
ALWAYS THINK SAFETY

Tunnel #4

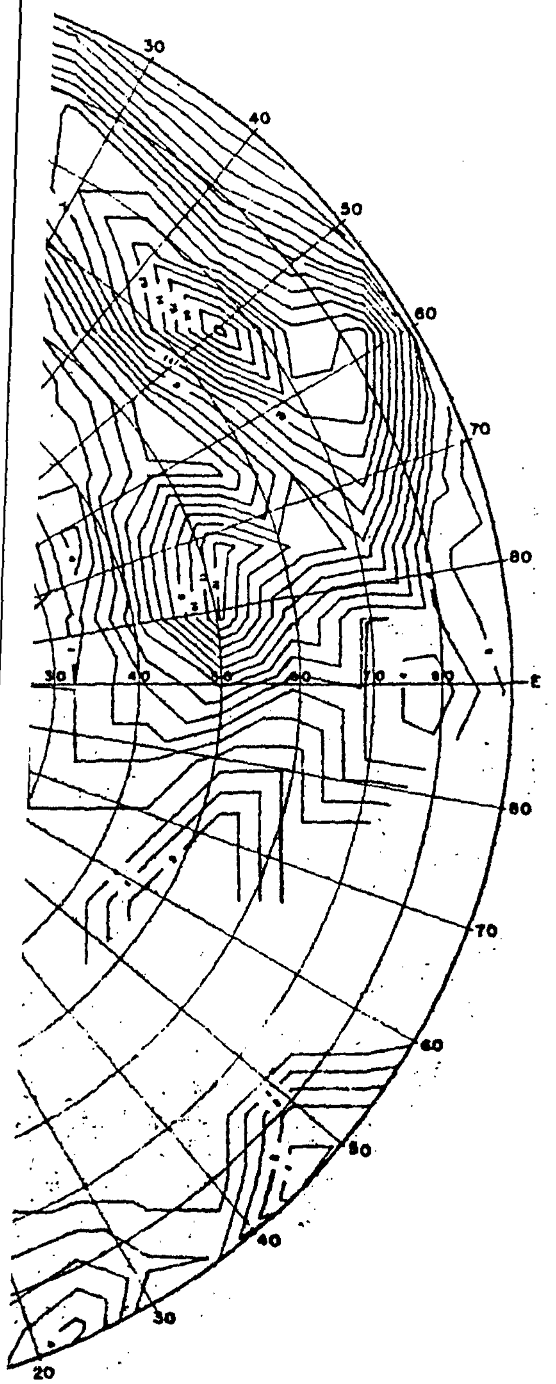
DRAWN ______ SUBMITTED ______
TRACED ______ RECOMMENDED ______
CHECKED ______ APPROVED ______

DENVER, COLORADO

2



DH-105 AUBURN DAM
3/29/68



Joints only

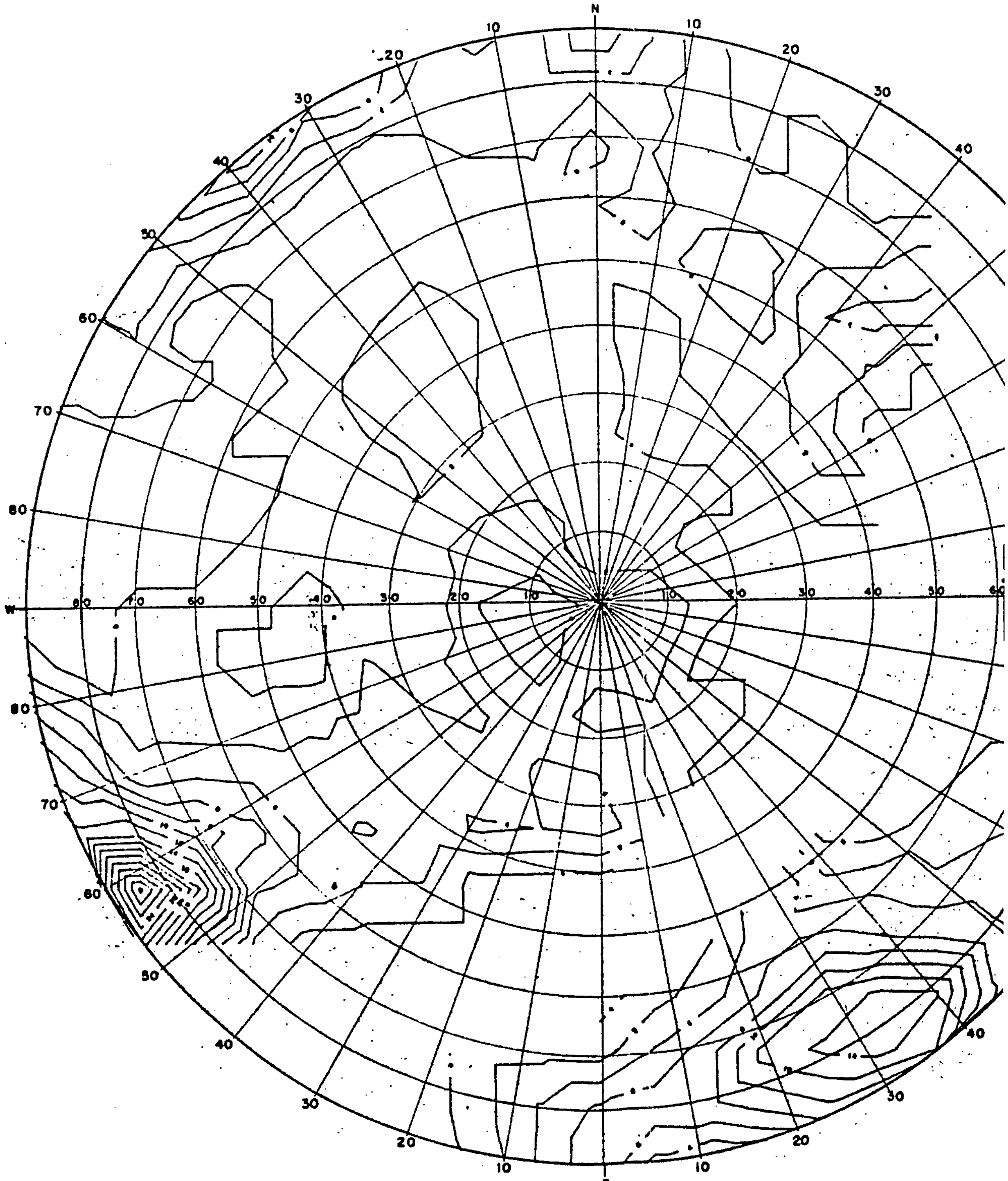
DH-105

DRAWN ______ SUBMITTED ______
TRACED ______ RECOMMENDED ______
CHECKED ______ APPROVED ______
DENVER, COLORADO

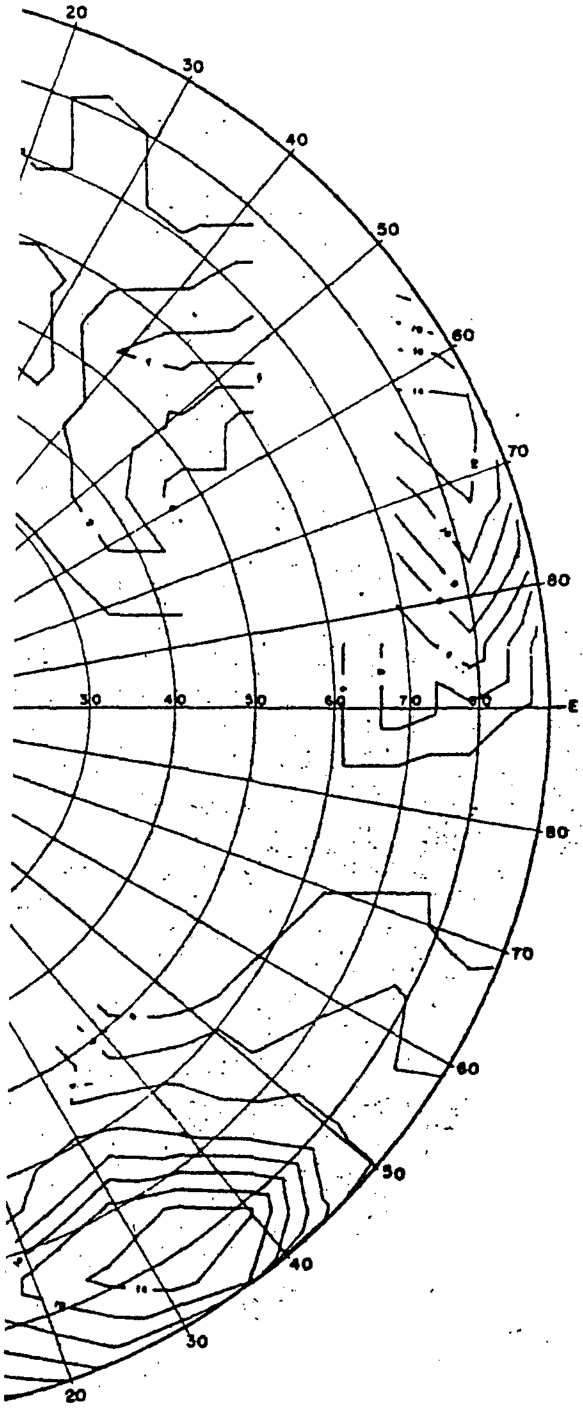
8-1-68

2

10 of 16



DH-107 AUBURN DAM
3/29/68



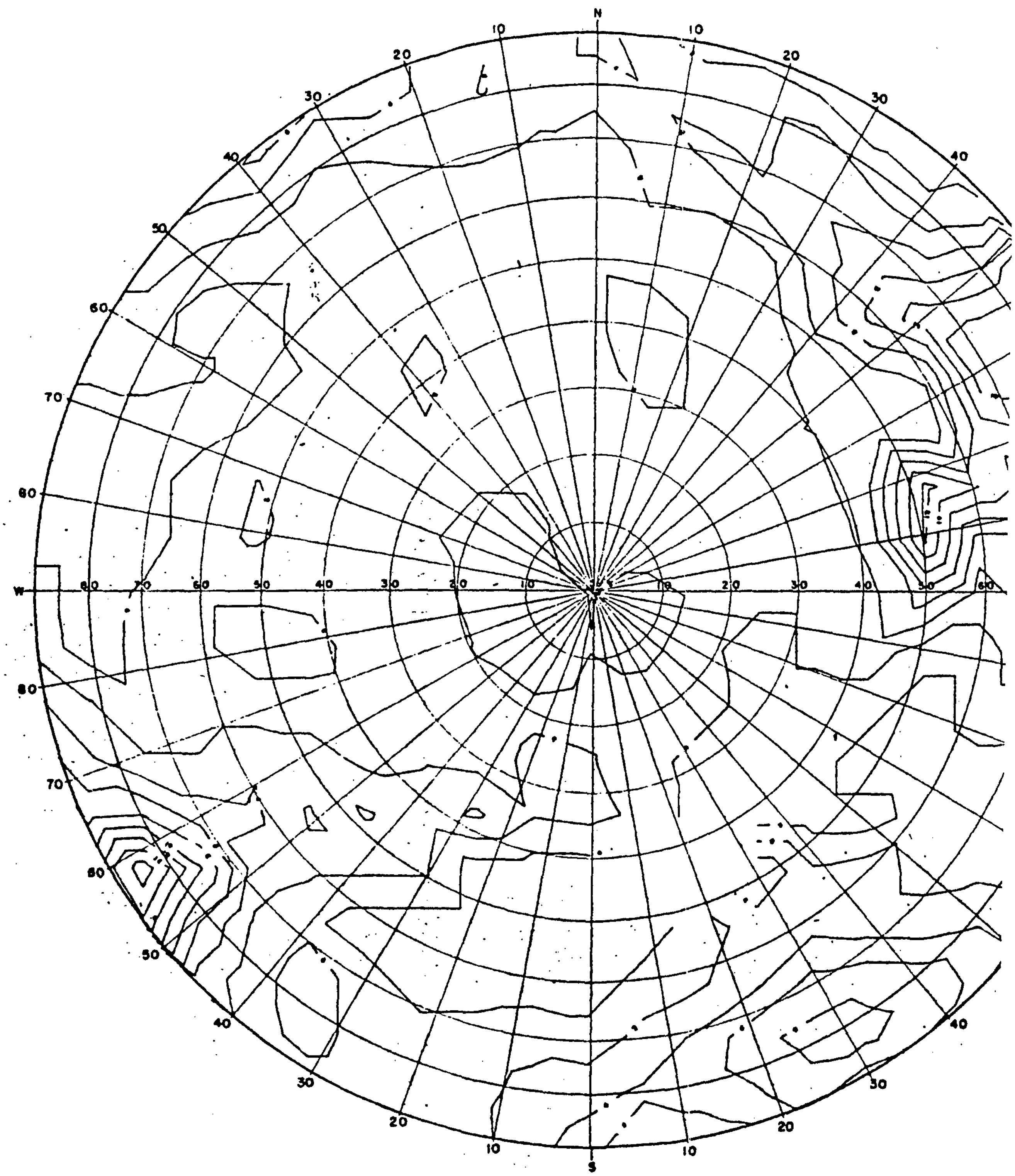
Joints only

ALWAYS THINK SAFETY

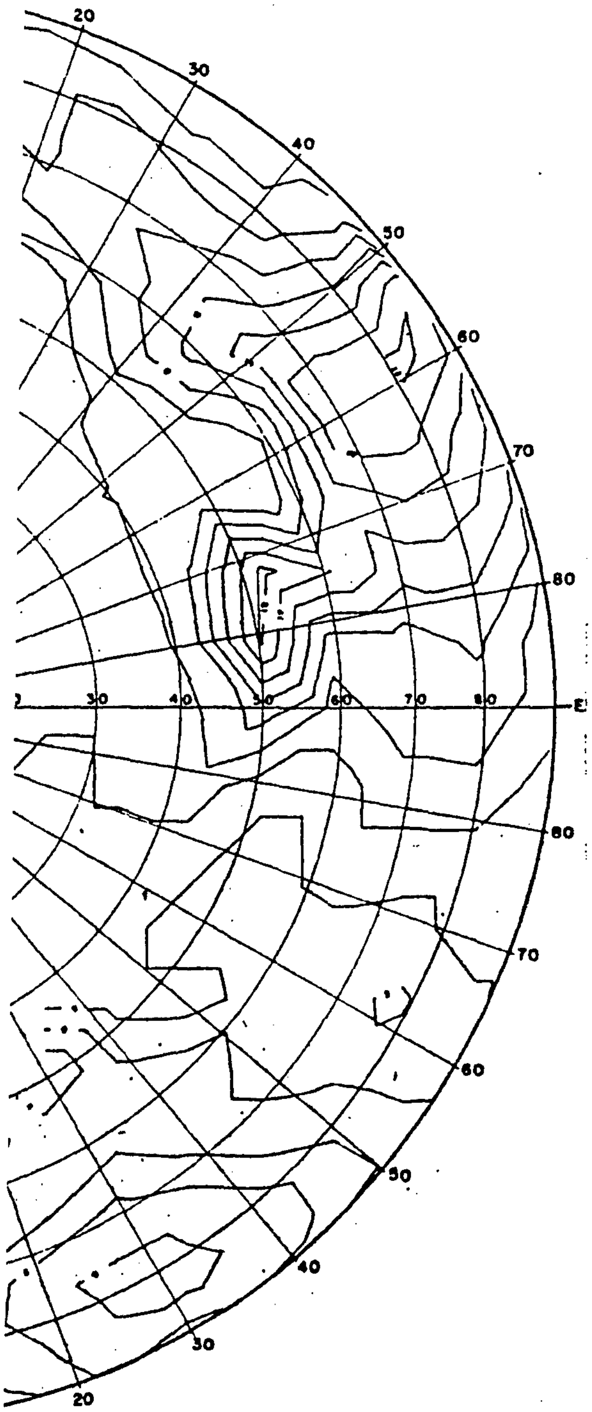
DH-107

DRAWN ______ SUBMITTED ______
TRACED ______ RECOMMENDED ______
CHECKED ______ APPROVED ______
DENVER, COLORADO

2



DH-105 AND DH-107 AUBURN DAM
07/24/68



Joints only

ALWAYS THINK SAFETY

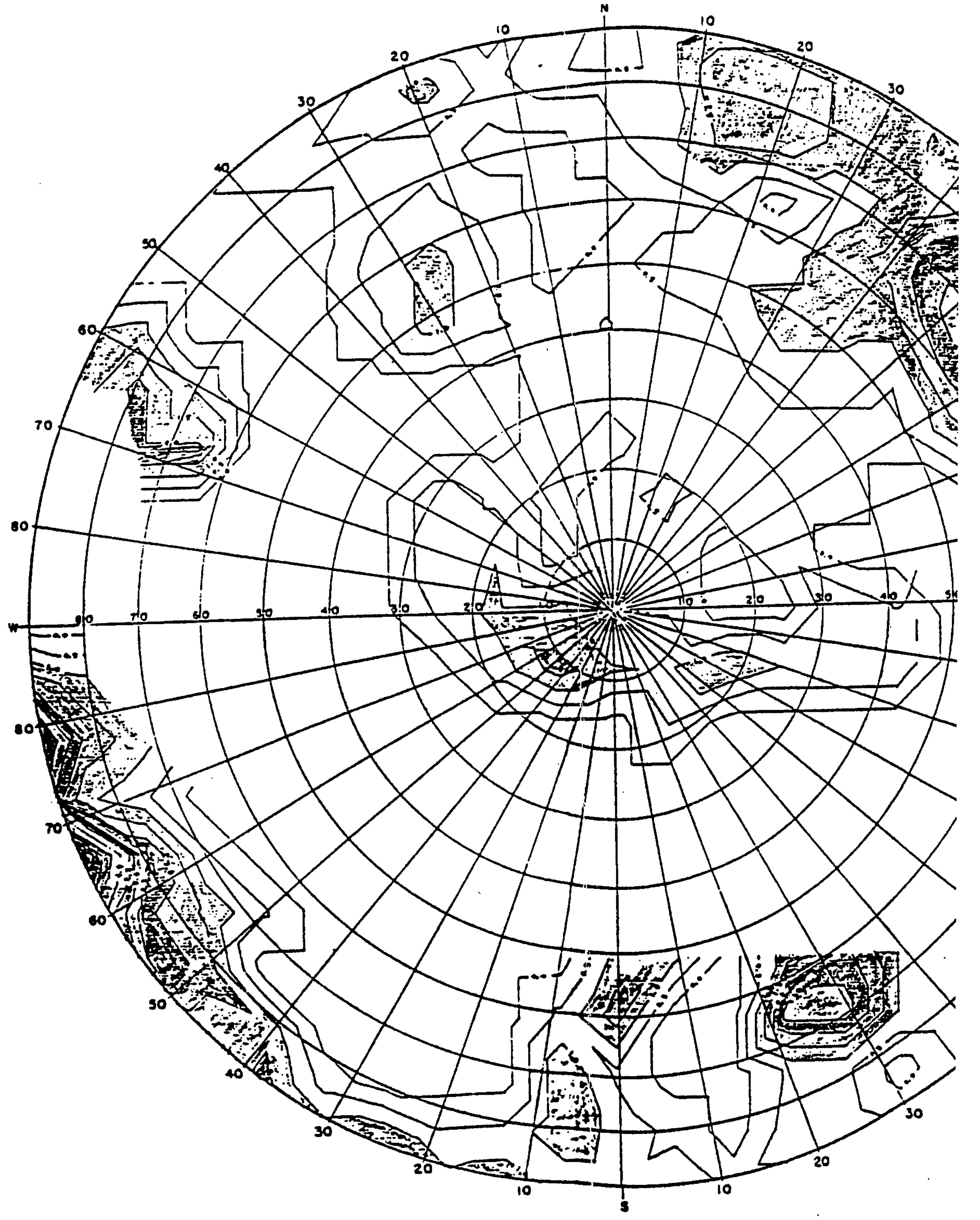
DH-105 + 107

DRAWN ______ SUBMITTED ______
TRACED ______ RECOMMENDED ______
CHECKED ______ APPROVED ______
DENVER, COLORADO

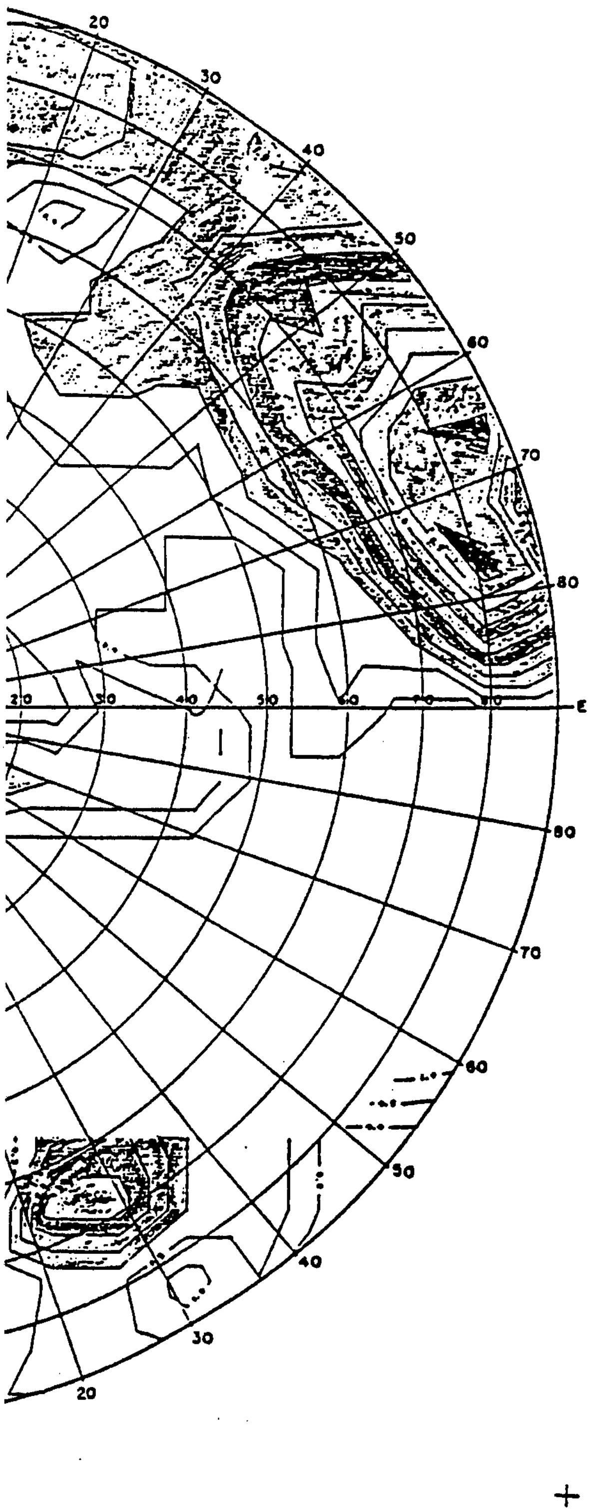
8-1-68

2

120716



CH-10S AUBURN DAM JOINTS
7/31/68



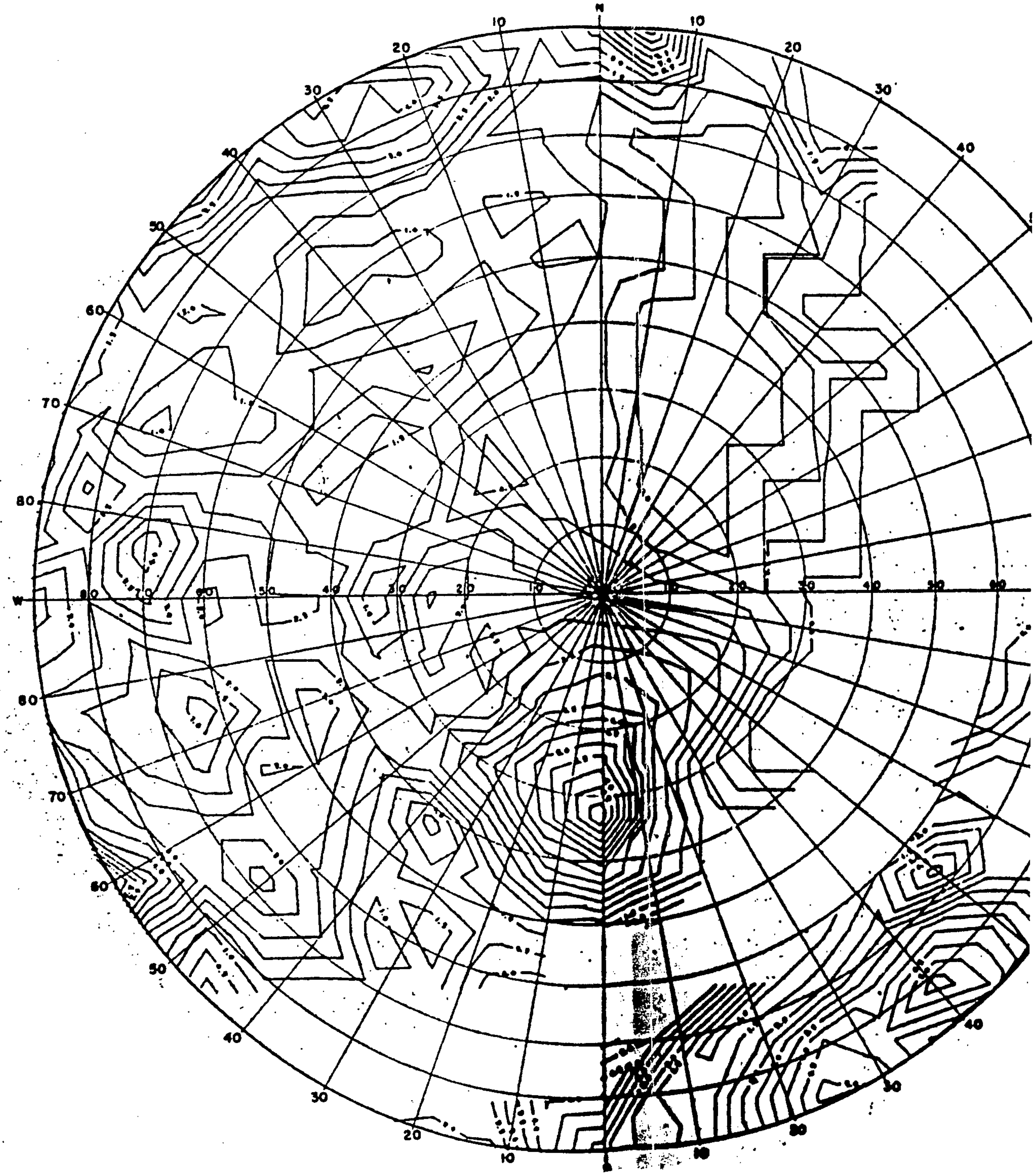
ALWAYS THINK SAFETY

DH-109

DENVER, COLORADO

2

130/16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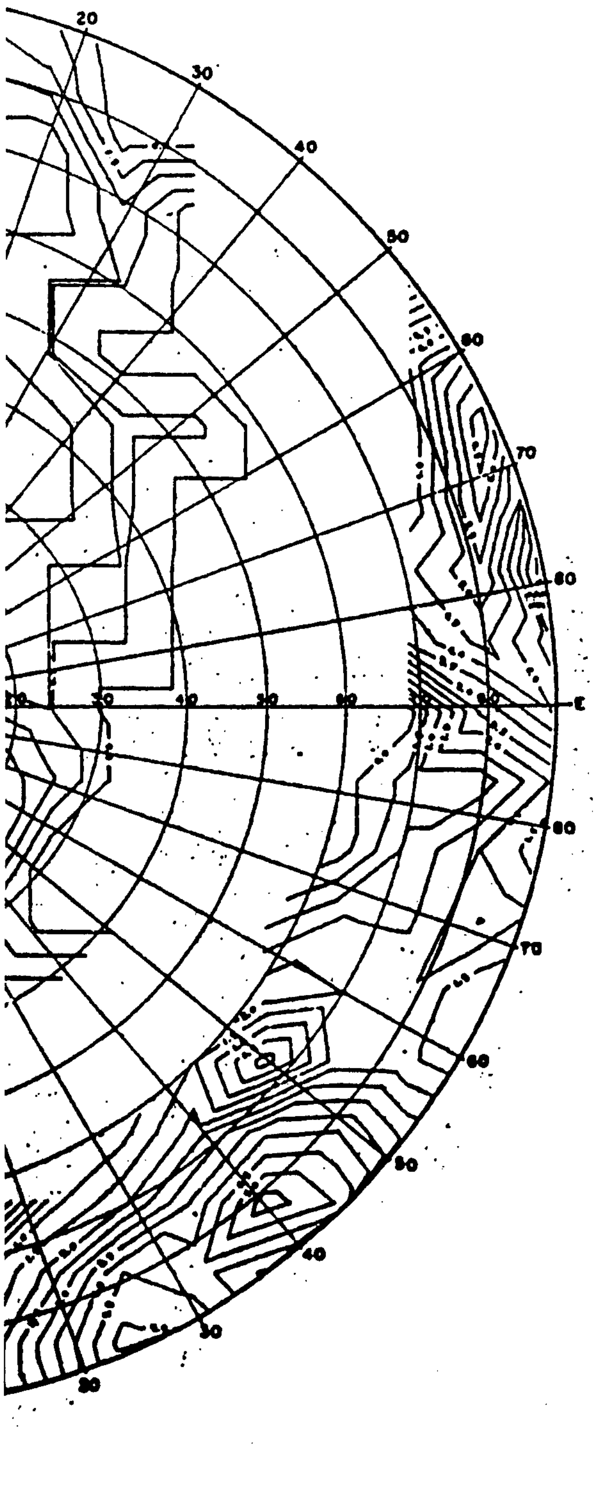
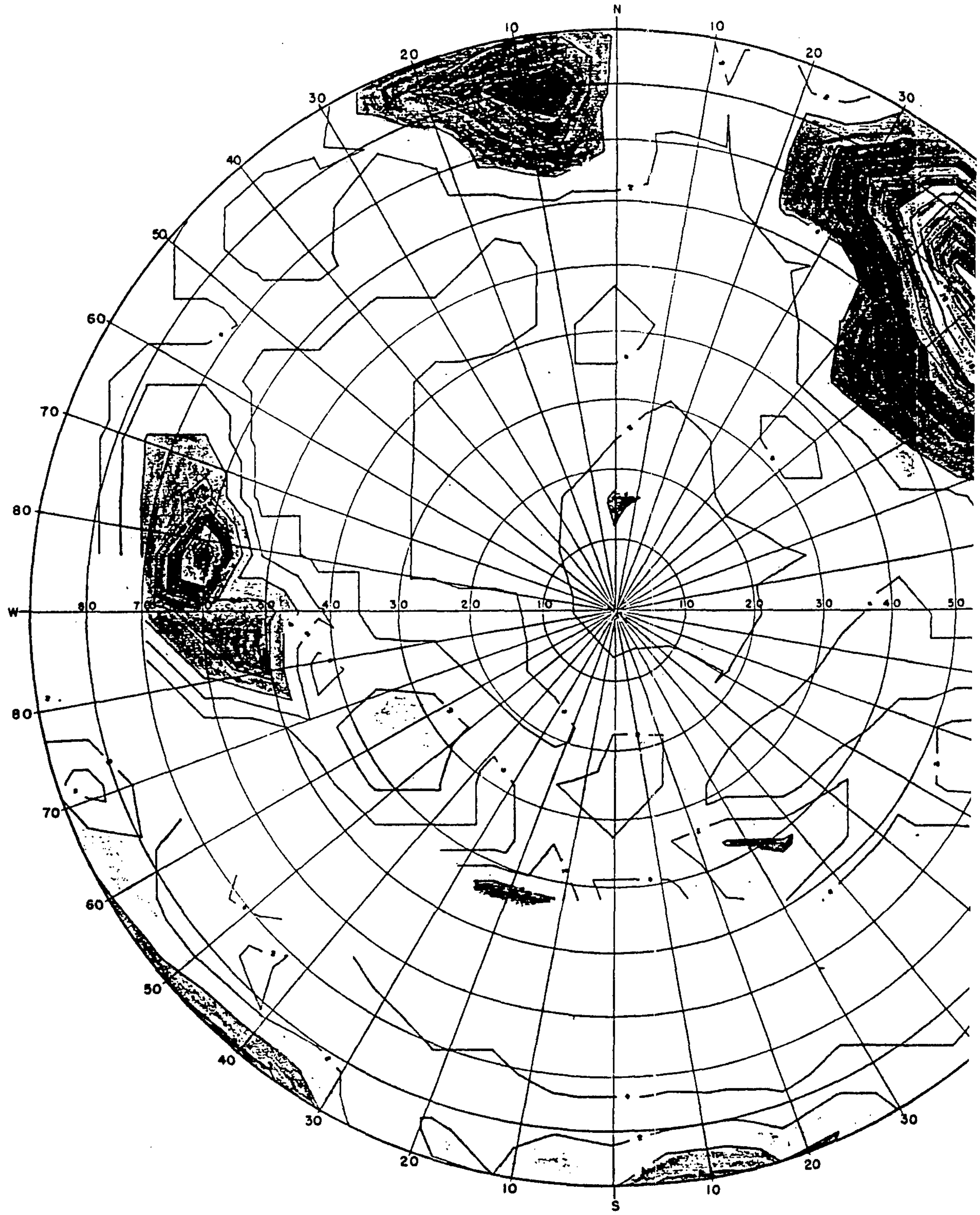
80

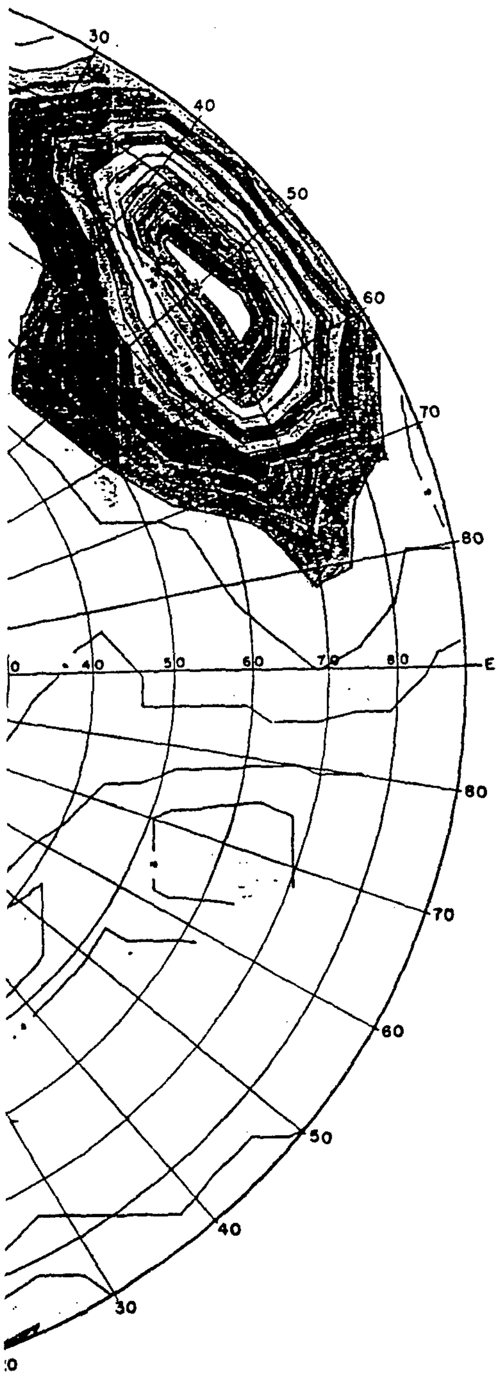


Joints, - shears, etc - not sorted



DH-111 AUBURN DAM JOINTS
7/31/68

Unclassified points = Blind zone = Depth interval = Run AD-14



ALWAYS THINK SAFETY

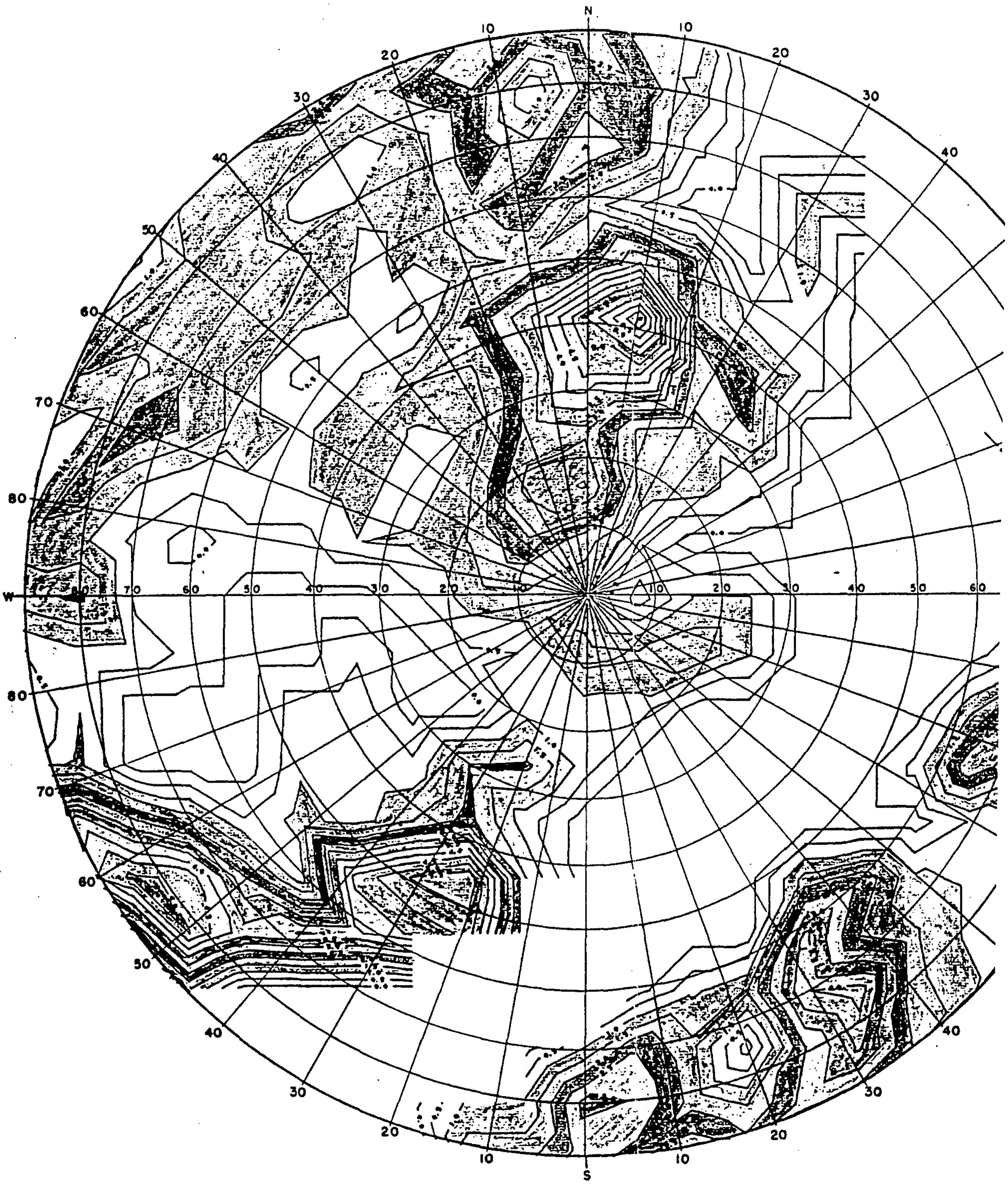
DH-111
JOINTS

DRAWN ________ SUBMITTED ________
TRACED ________ RECOMMENDED ________
CHECKED ________ APPROVED ________
DENVER, COLORADO

2

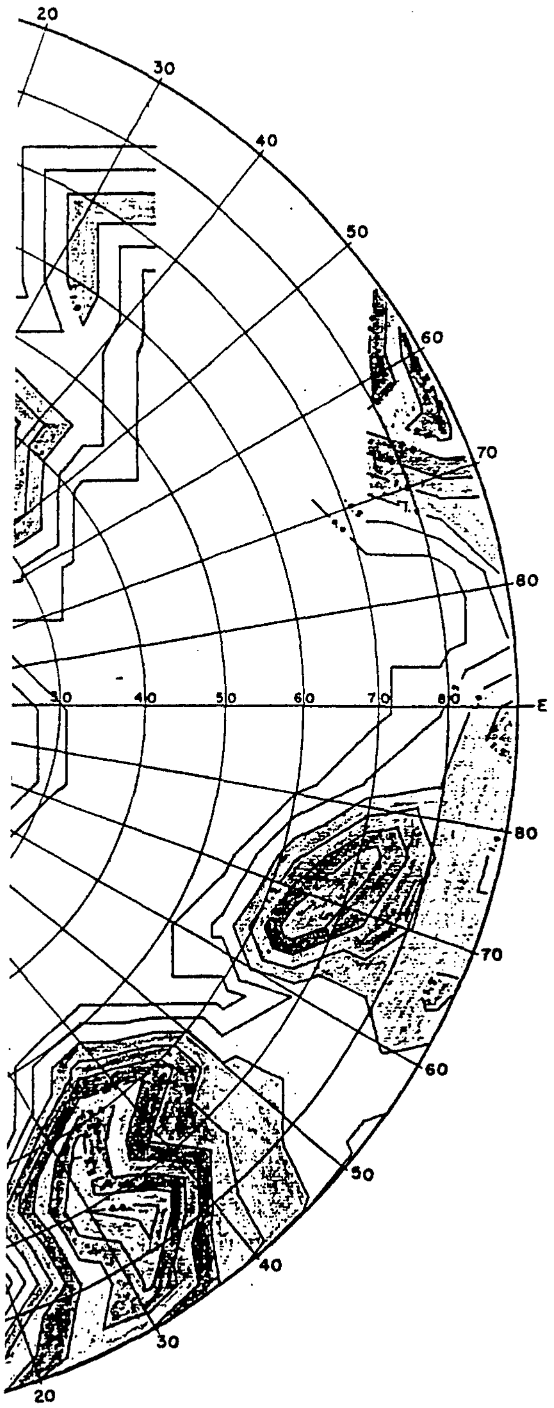
15 of 16

Unclassified joints



DH 113   AUBURN DAM   JOINTS
8/ 9/68

Unclassified points Blind zone = Depth interval = Run AD-15



ALWAYS THINK SAFETY

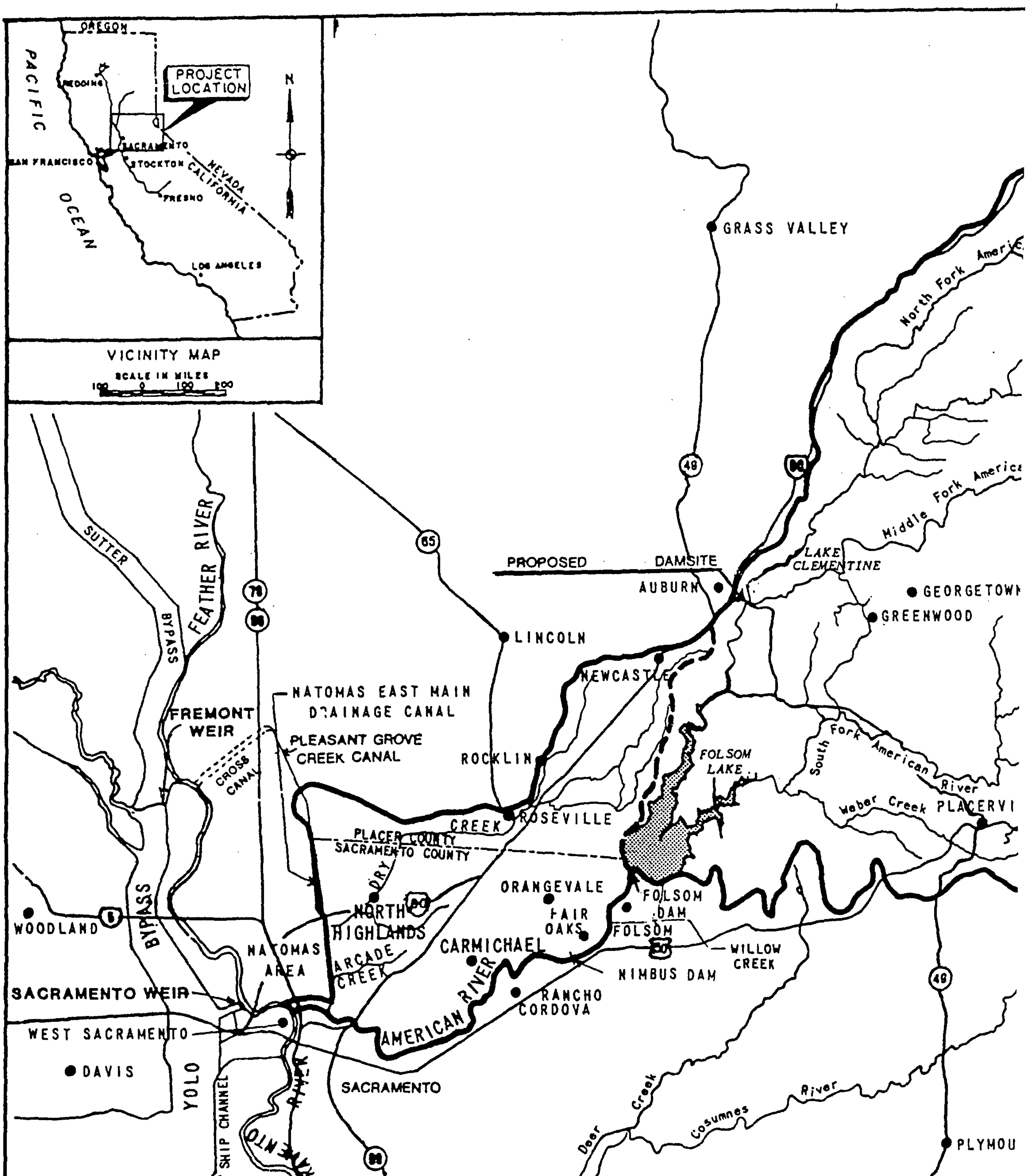
DH - 113

DRAWN ____ SUBMITTED ____
TRACED ____ RECOMMENDED ____
CHECKED ____ APPROVED ____

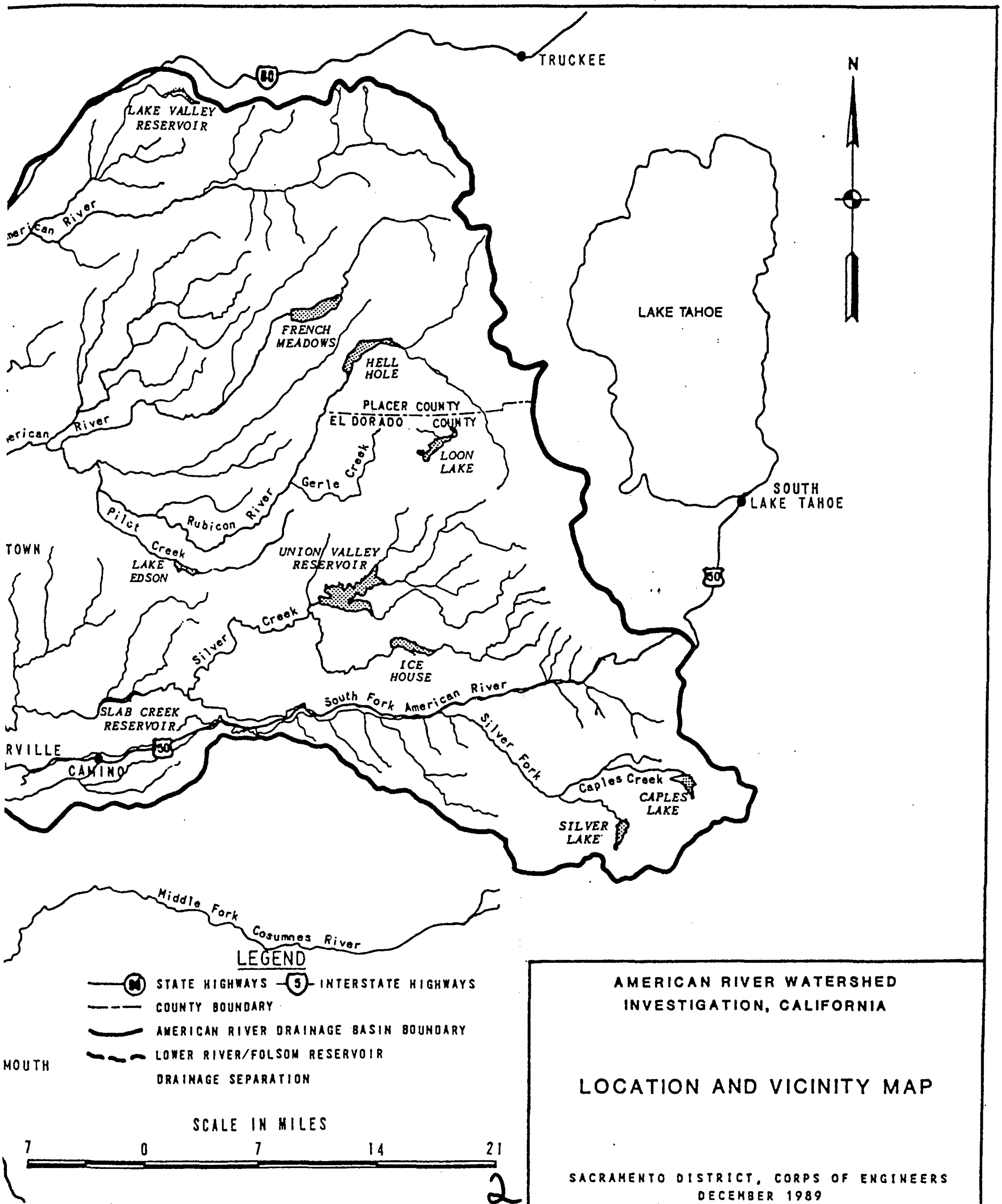
DENVER, COLORADO.

2

160+16



OREGON
PACIFIC
OCEAN
PROJECT LOCATION
N
REDDING
SACRAMENTO
SAN FRANCISCO
STOCKTON
NEVADA
CALIFORNIA
FRESNO
LOS ANGELES
VICINITY MAP
SCALE IN MILES
100 0 100 200
GRASS VALLEY
North Fork American
Middle Fork American
49
80
65
PROPOSED DAMSITE
LAKE CLEMENTINE
AUBURN
GEORGETOWN
GREENWOOD
LINCOLN
NEWCASTLE
SUTTER BYPASS
FEATHER RIVER
70
99
FREMONT WEIR
NATOMAS EAST MAIN DRAINAGE CANAL
PLEASANT GROVE CREEK CANAL
CROSS CANAL
ROCKLIN
FOLSOM LAKE
South Fork American River
Weber Creek
PLACERVI
CREEK
ROSEVILLE
PLACER COUNTY
SACRAMENTO COUNTY
DRY
ORANGEVALE
FOLSOM DAM
WOODLAND
6
BYPASS
NORTH HIGHLANDS
FAIR OAKS
FOLSOM
WILLOW CREEK
NATOMAS AREA
ARCADE CREEK
CARMICHAEL
50
NIMBUS DAM
SACRAMENTO WEIR
AMERICAN RIVER
RANCHO CORDOVA
WEST SACRAMENTO
DAVIS
YOLO
DEEP WATER SHIP CHANNEL
SACRAMENTO RIVER
SACRAMENTO
Deer Creek
Cosumnes River
PLYMOU
SHELDON RD
5
ELK GROVE

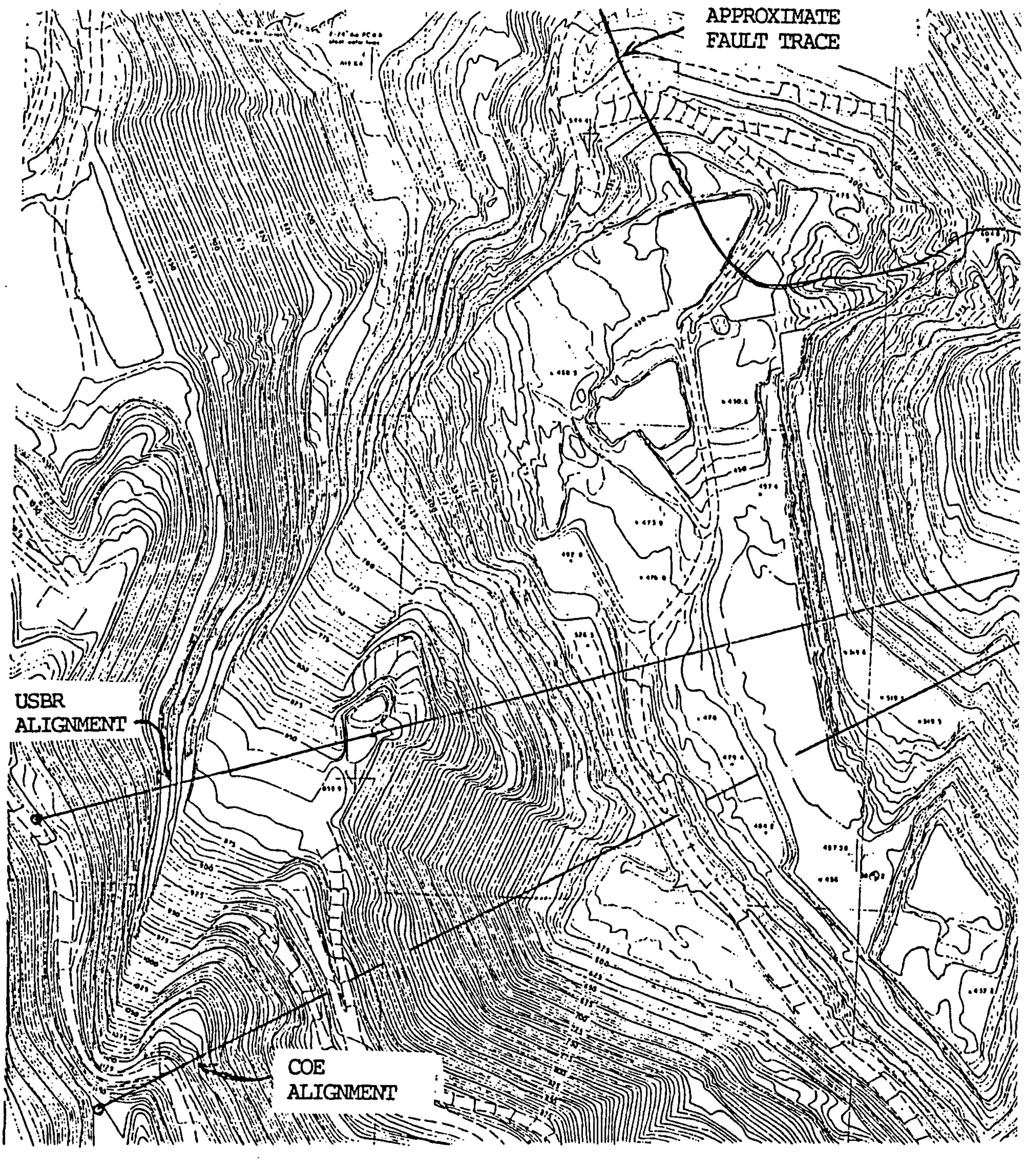


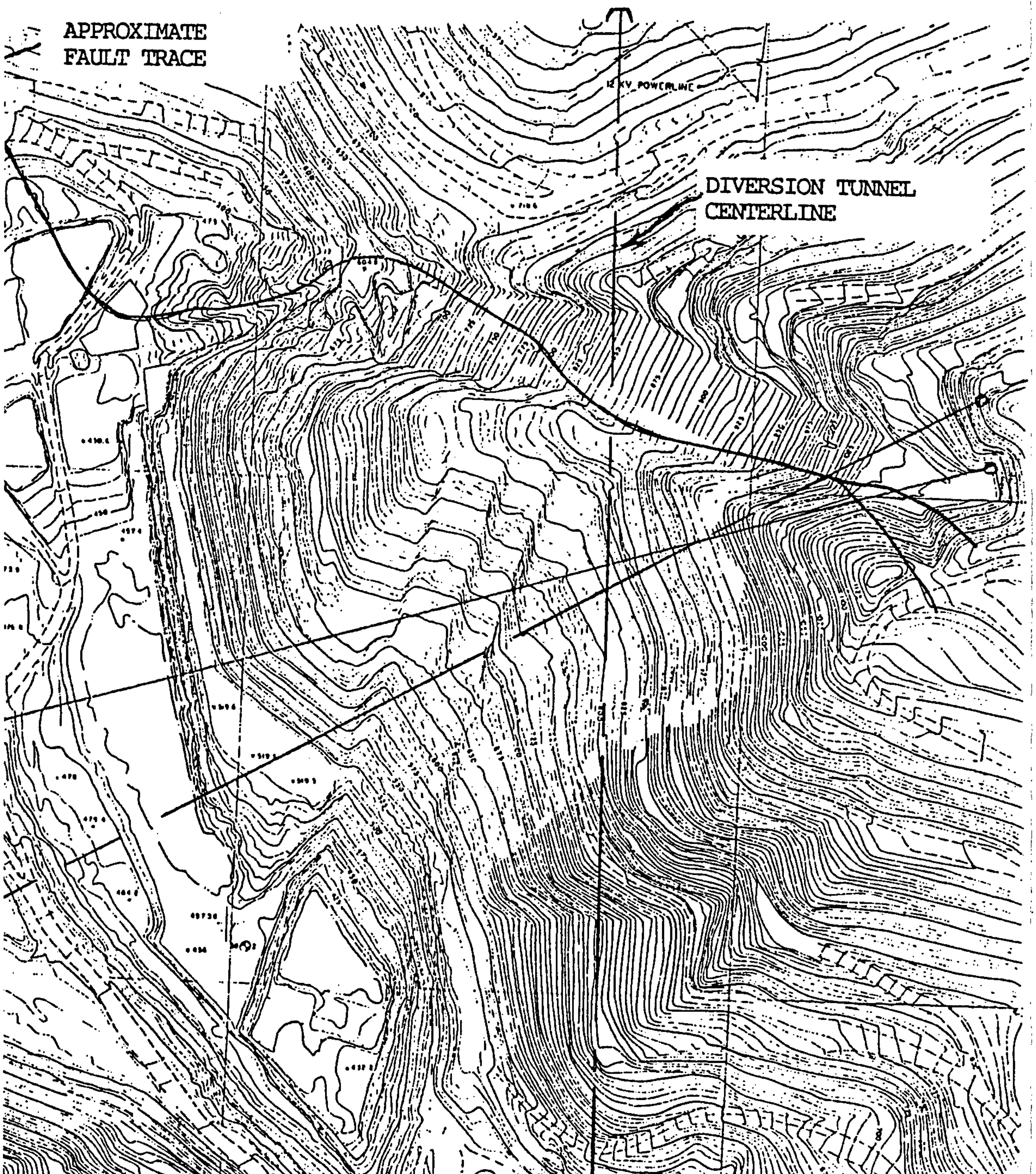
AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

LOCATION AND VICINITY MAP

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
DECEMBER 1989

PLATE 1


APPROXIMATE
FAULT TRACE
USBR
ALIGNMENT
COE
ALIGNMENT



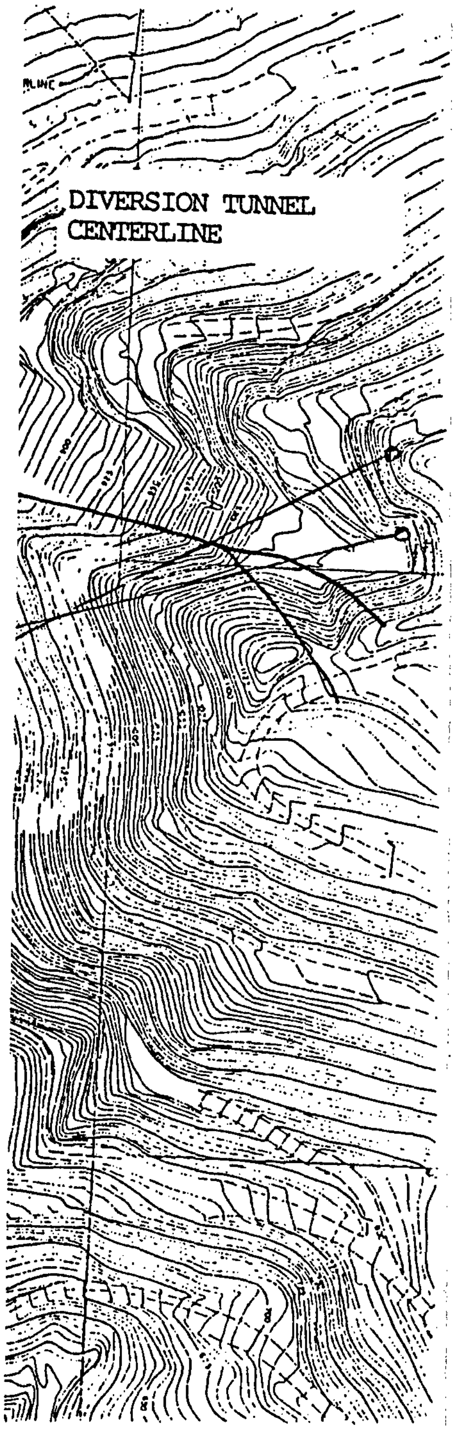
AMERICAN RIVER WA
CA

DAM ALT

ALIG

SACRAMENTO DISTRICT
MA

2



3

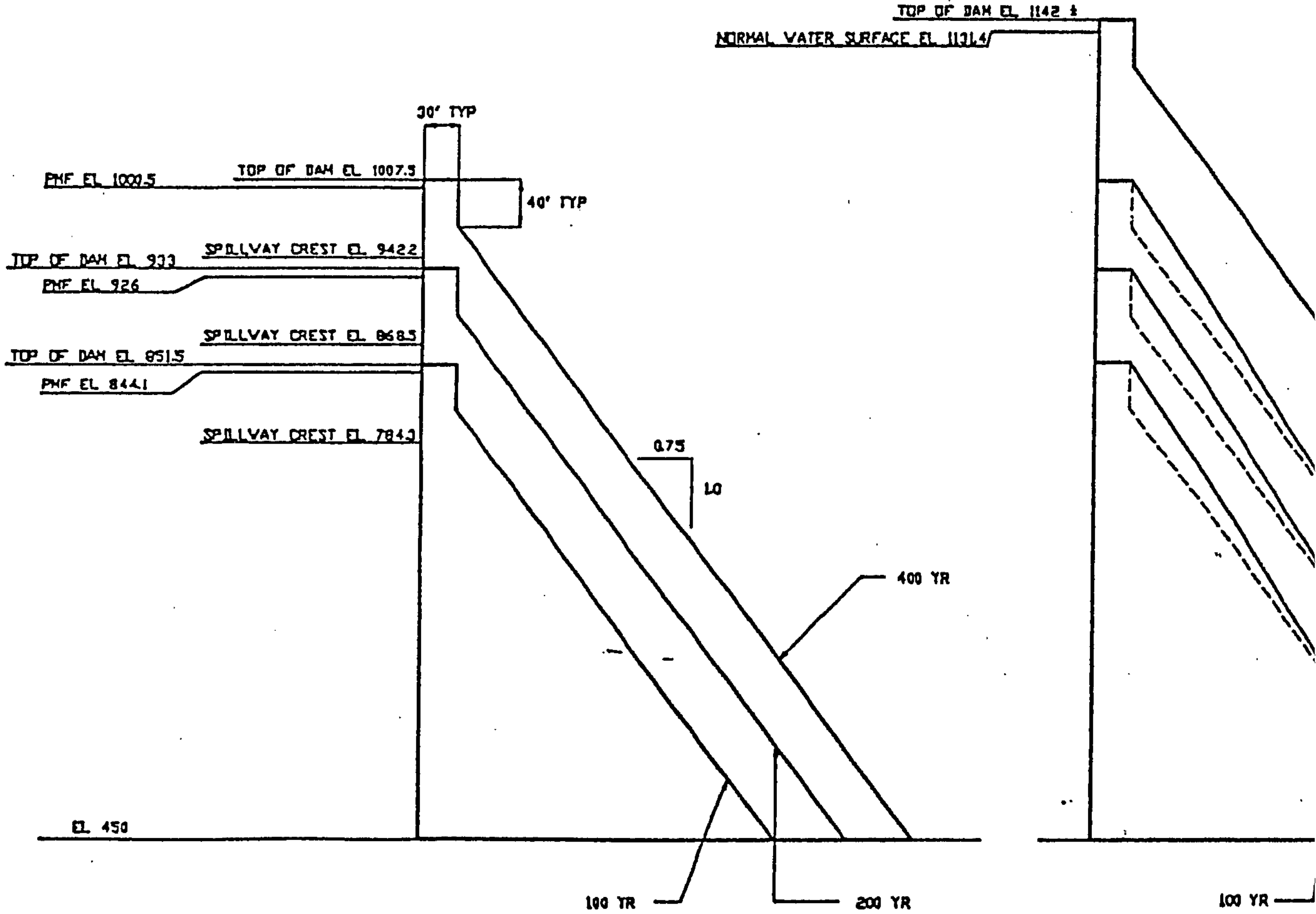
AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

DAM ALTERNATIVES

ALIGNMENT

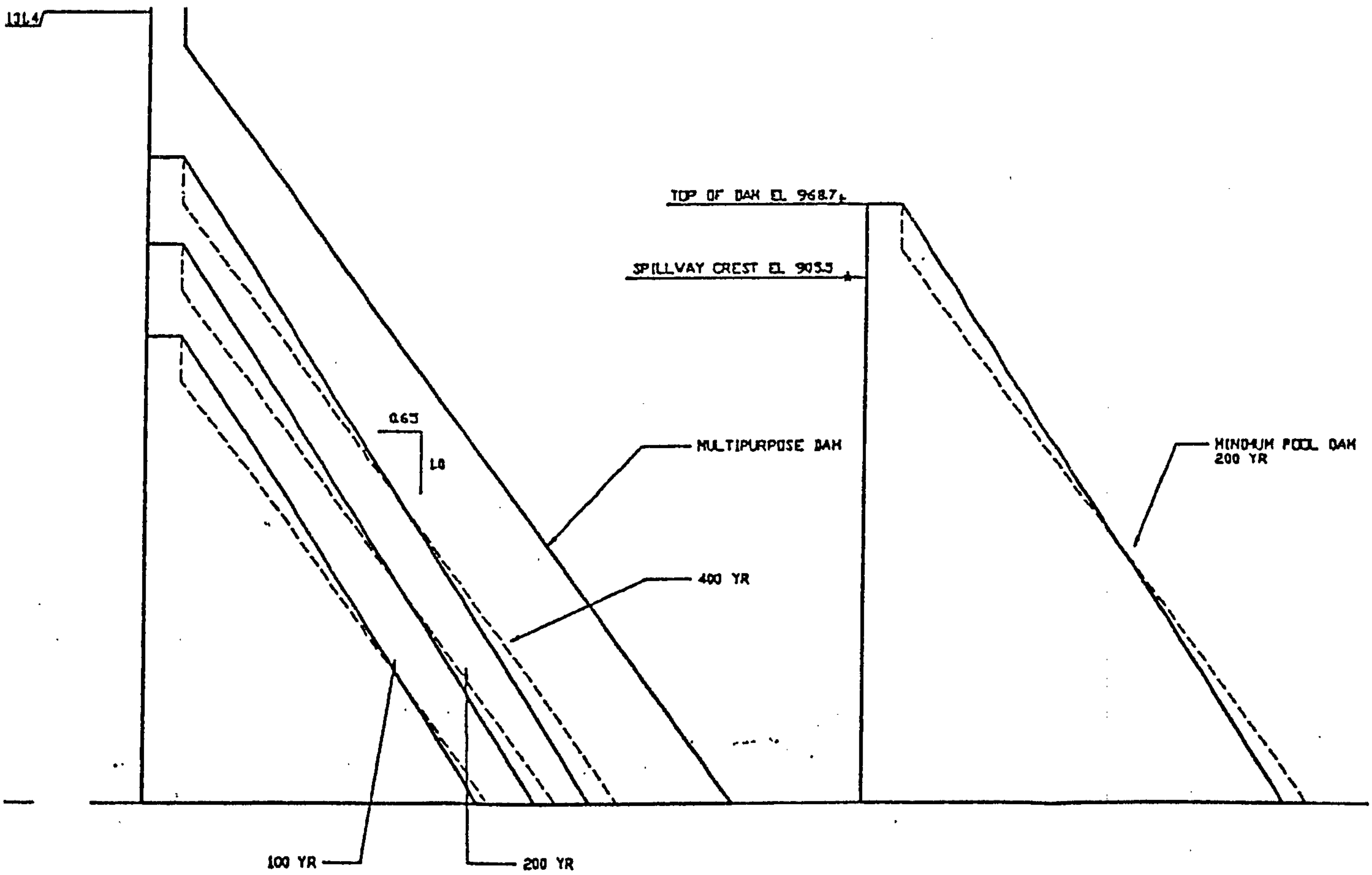
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 2



NONEXPANDABLE RCC DAMS
(FLOOD CONTROL ONLY)

DAM FEASIBI



EXPANDABLE RCC DAMS

DAM FEASIBILITY STUDIES

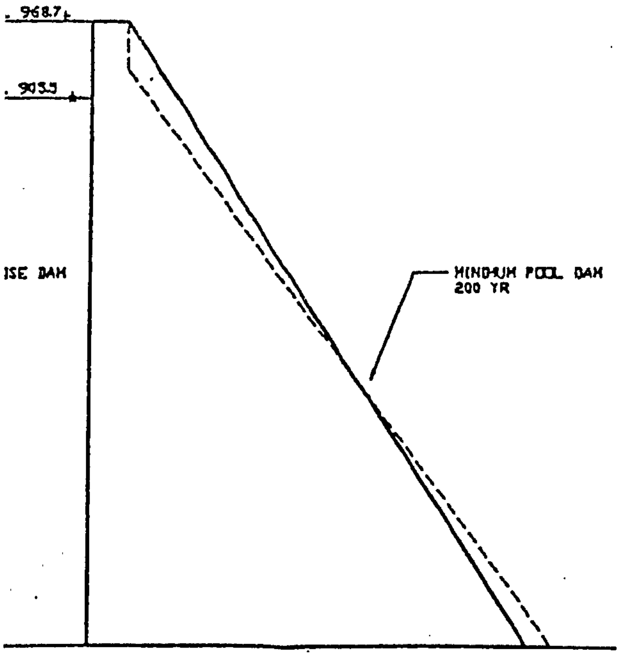
AMERICAN RIVER WATERS
CALIFO

DAM ALTE

NON-OVERFLO

SACRAMENTO DISTRICT, CC
MARCH

2



AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

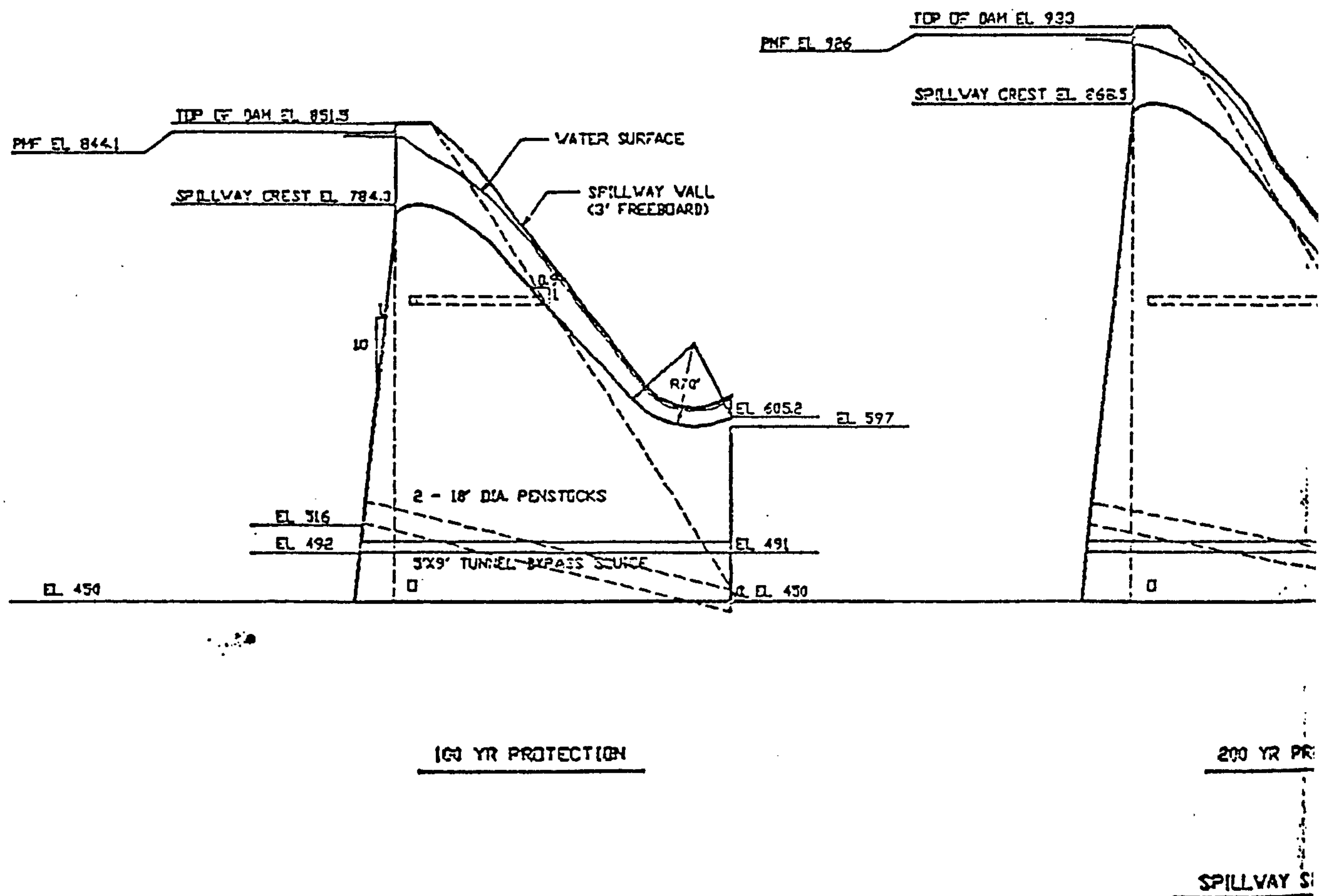
DAM ALTERNATIVES

NON-OVERFLOW SECTIONS

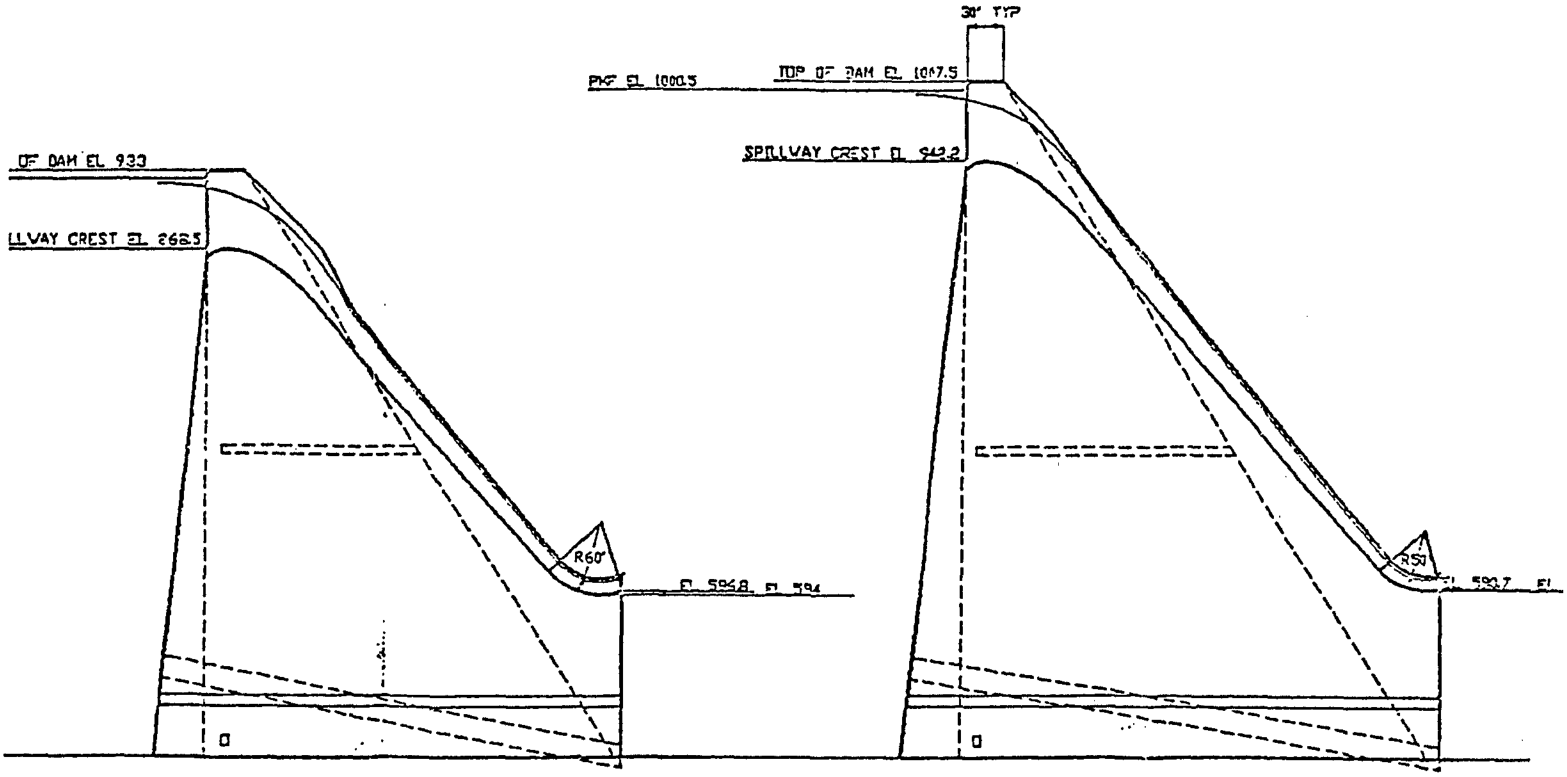
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

3

PLATE 3



DAM FEASIBILITY



200 YR PROTECTION

400 YR PROTECTION

SPILLWAY SECTIONS

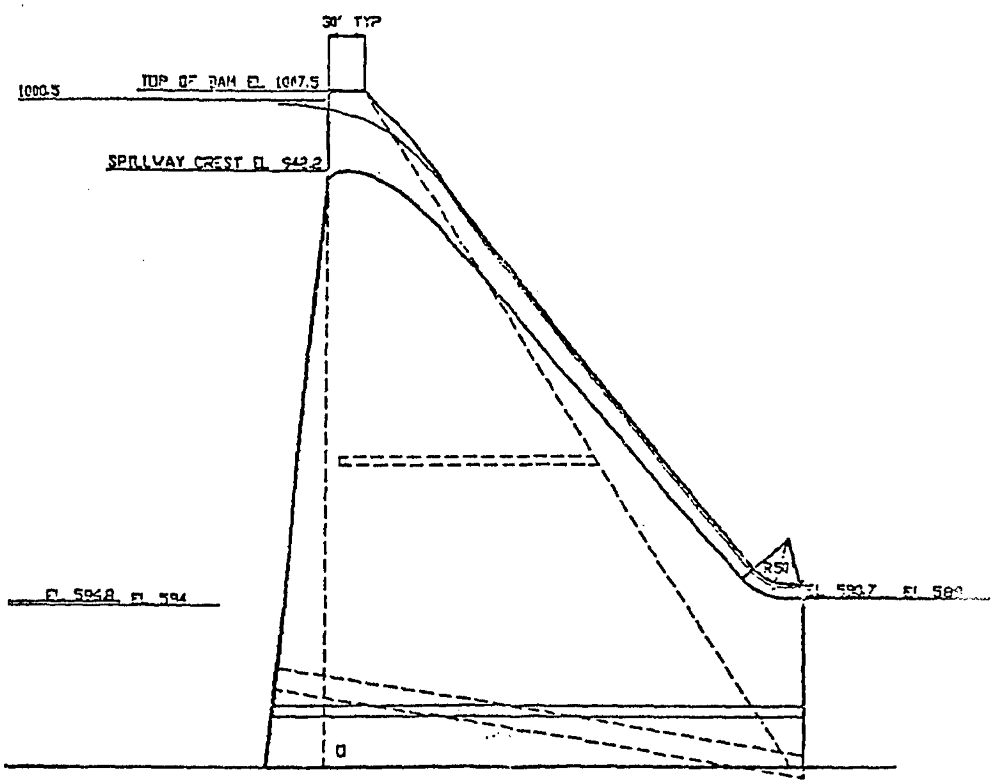
DAM FEASIBILITY STUDIES

AMERICAN RIVER WATERS
CALIFO

DAM ALTER

SPILLWAY

SACRAMENTO DISTRICT, C
MARCH



400 YR PROTECTION

AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

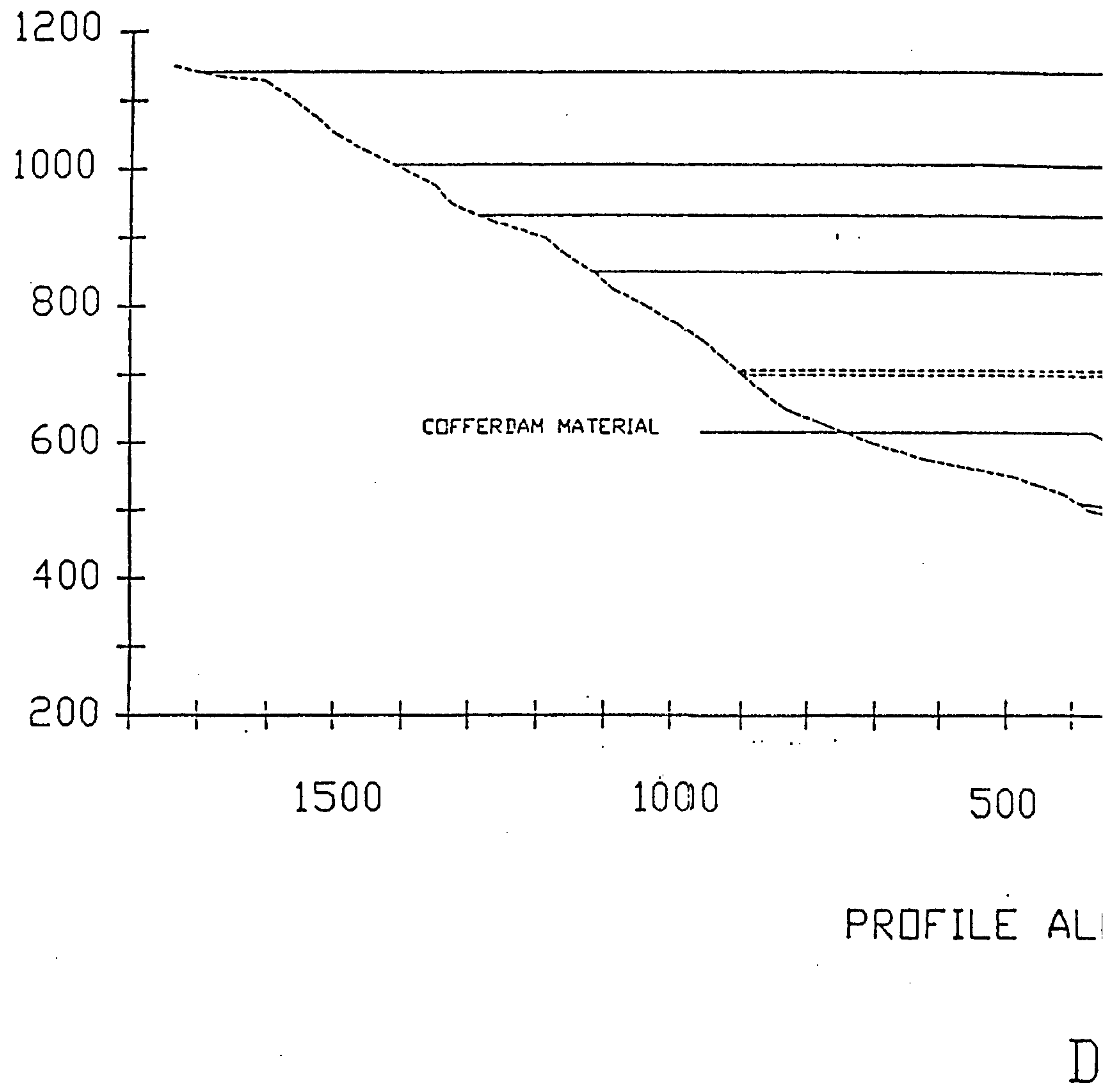
DAM ALTERNATIVES

SPILLWAY SECTIONS

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

3

PLATE 4



1200
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COFFERDAM MATERIAL
1500
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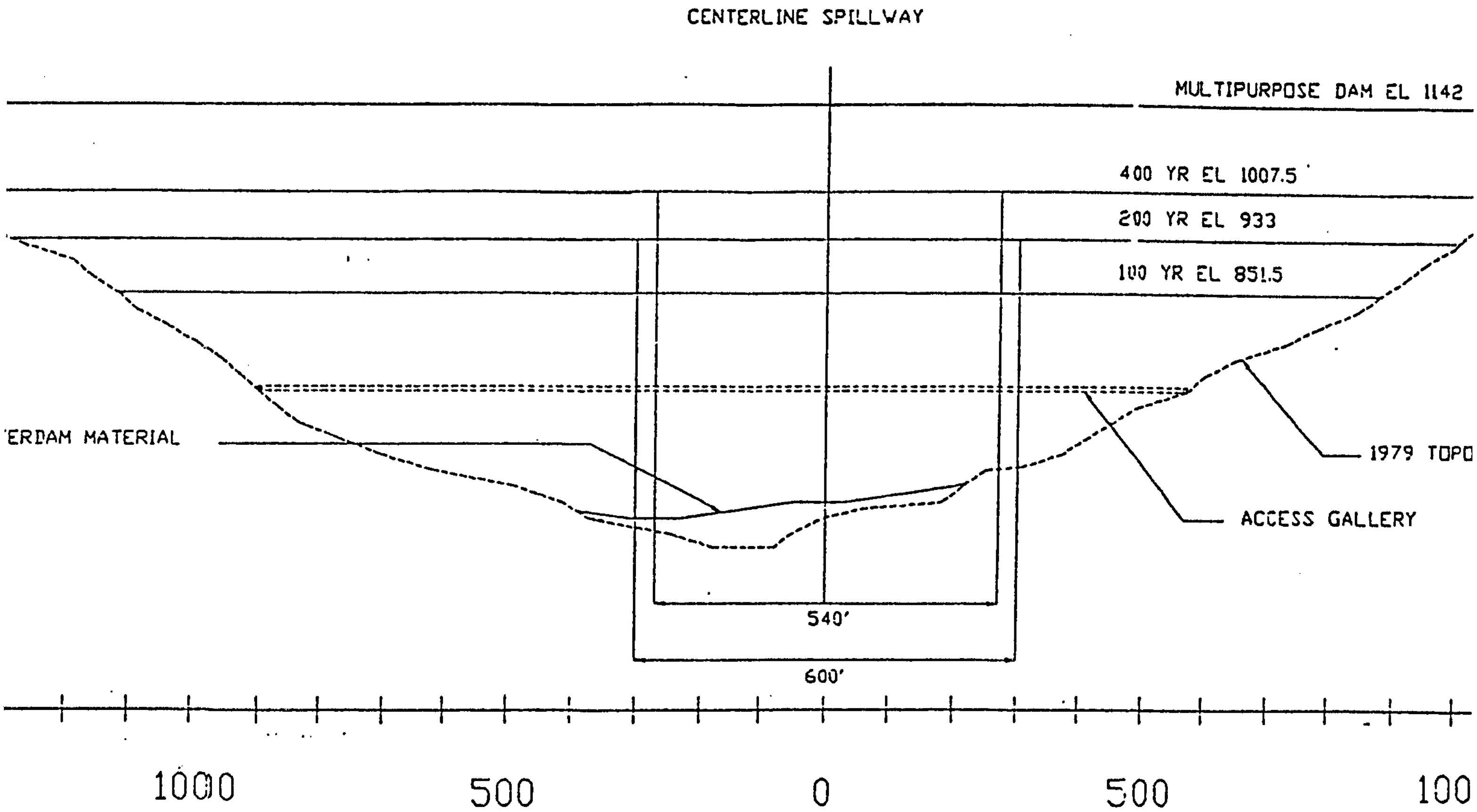
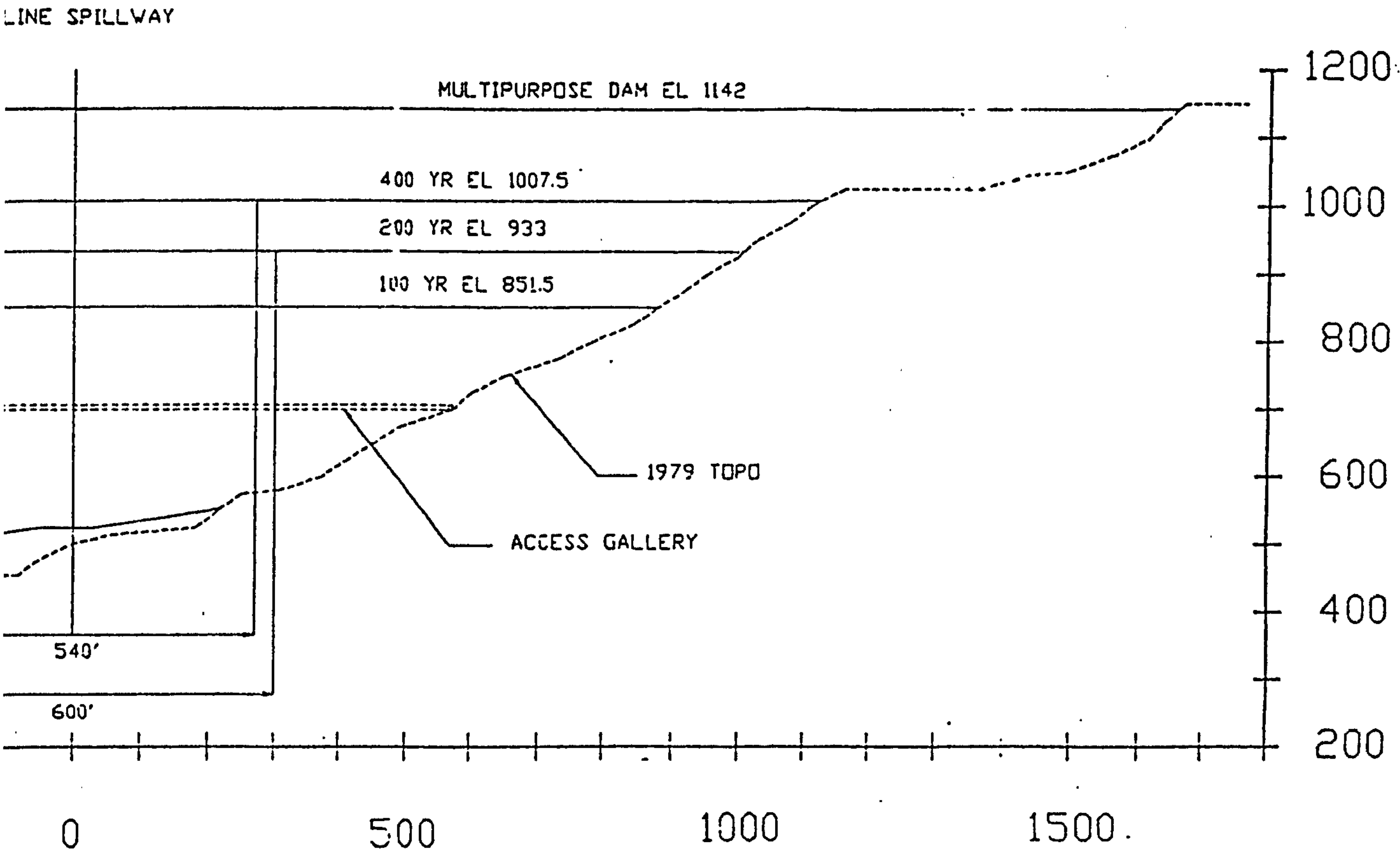
PROFILE AL
D



PROFILE ALONG CENTERLINE OF TOP OF DAM
(LOOKING UPSTREAM)

DAM FEASIBILITY STUDIES



NTERLINE OF TOP OF DAM
IKING UPSTREAM)

EASIBILITY STUDIES

AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

DAM ALTERNATIVES

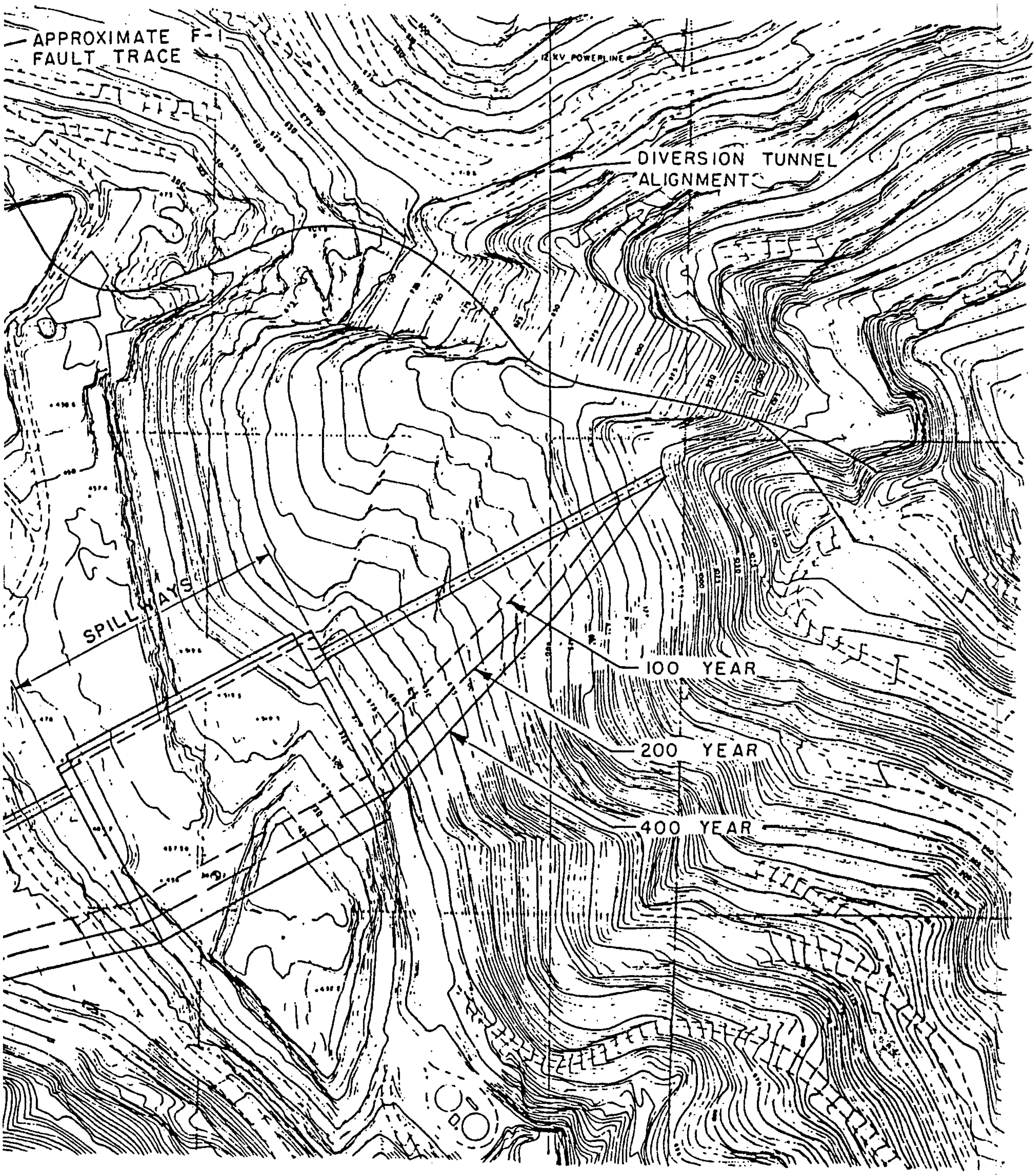
ELEVATIONS

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

3

PLATE 5

1



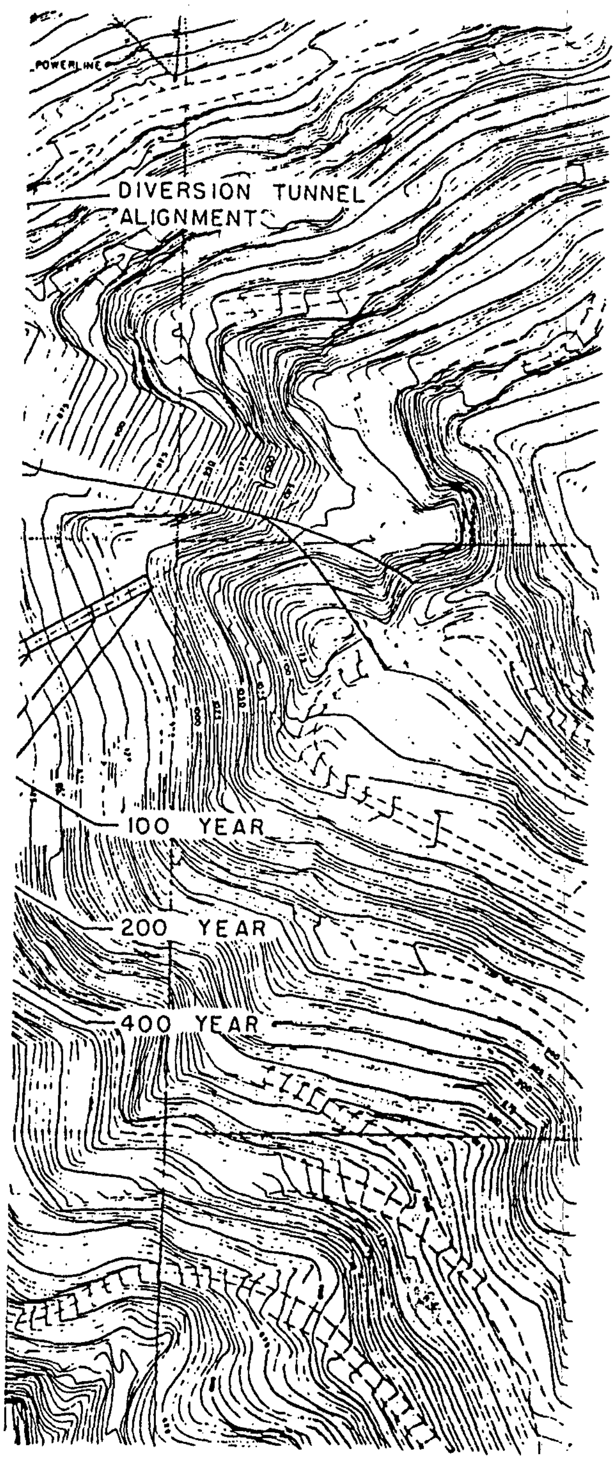
AMERICAN RIVER

DAM

P

SACRAMENTO DIST

2



3

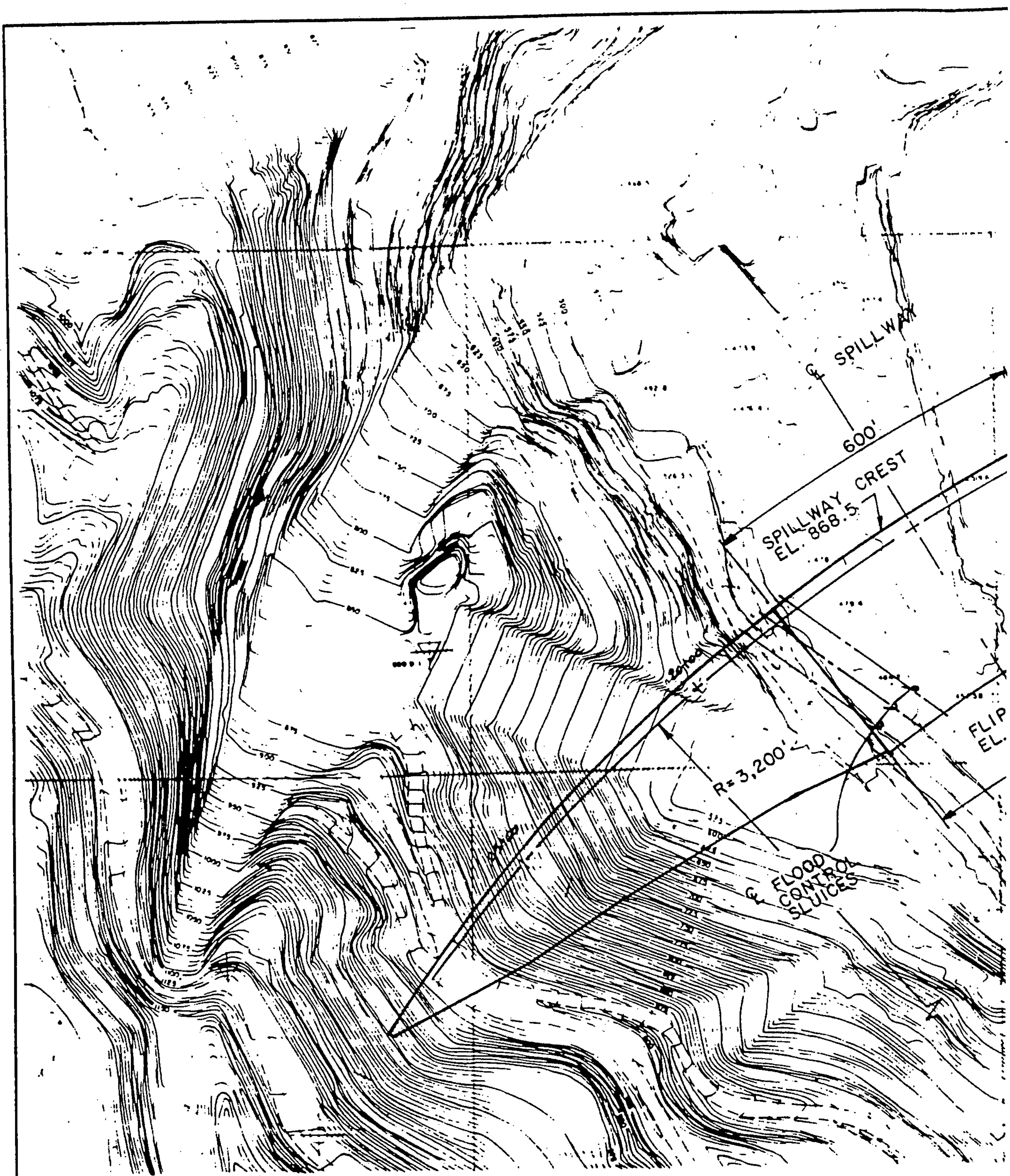
AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

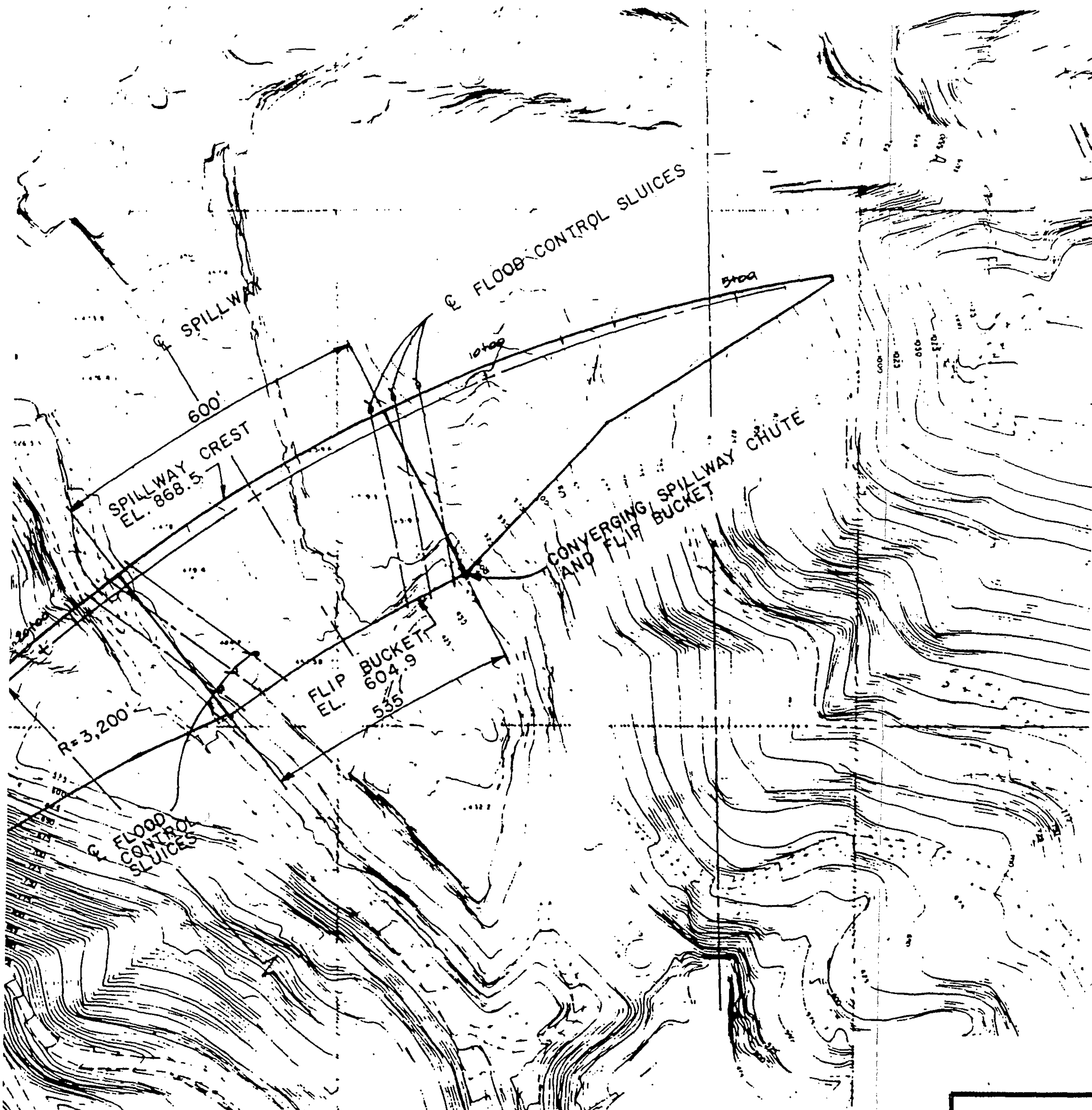
DAM ALTERNATIVES

PLAN VIEWS

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
MARCH 1990

PLATE 6

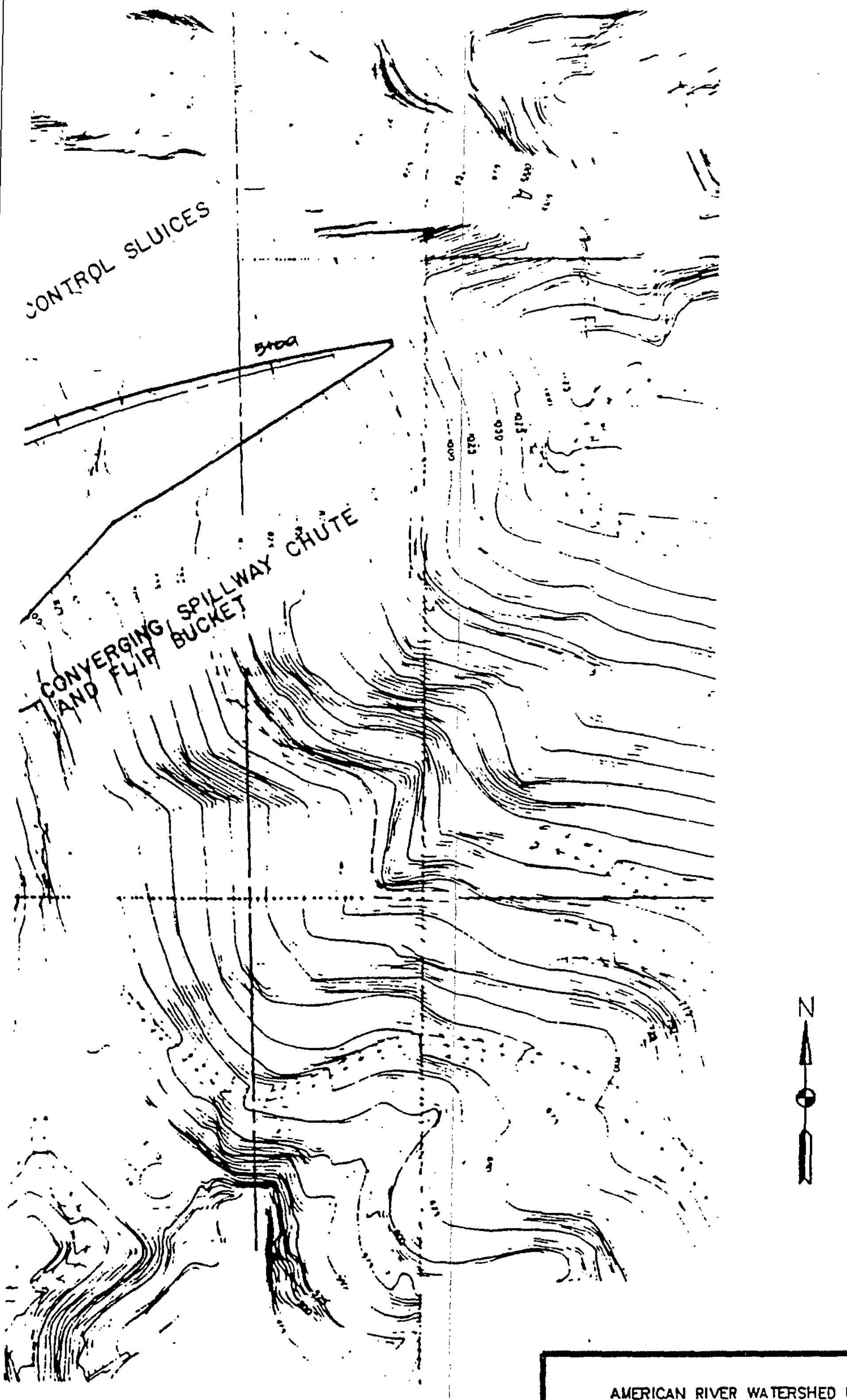

℄ SPILLWAY
600'
SPILLWAY CREST
EL. 868.5
R=3,200'
℄ FLOOD CONTROL SLUICES
FLIP
EL.



AMERICAN RIVER

SE
DAM

DESIGNED: RLP
DRAWN: GEB
CHECKED: PLP

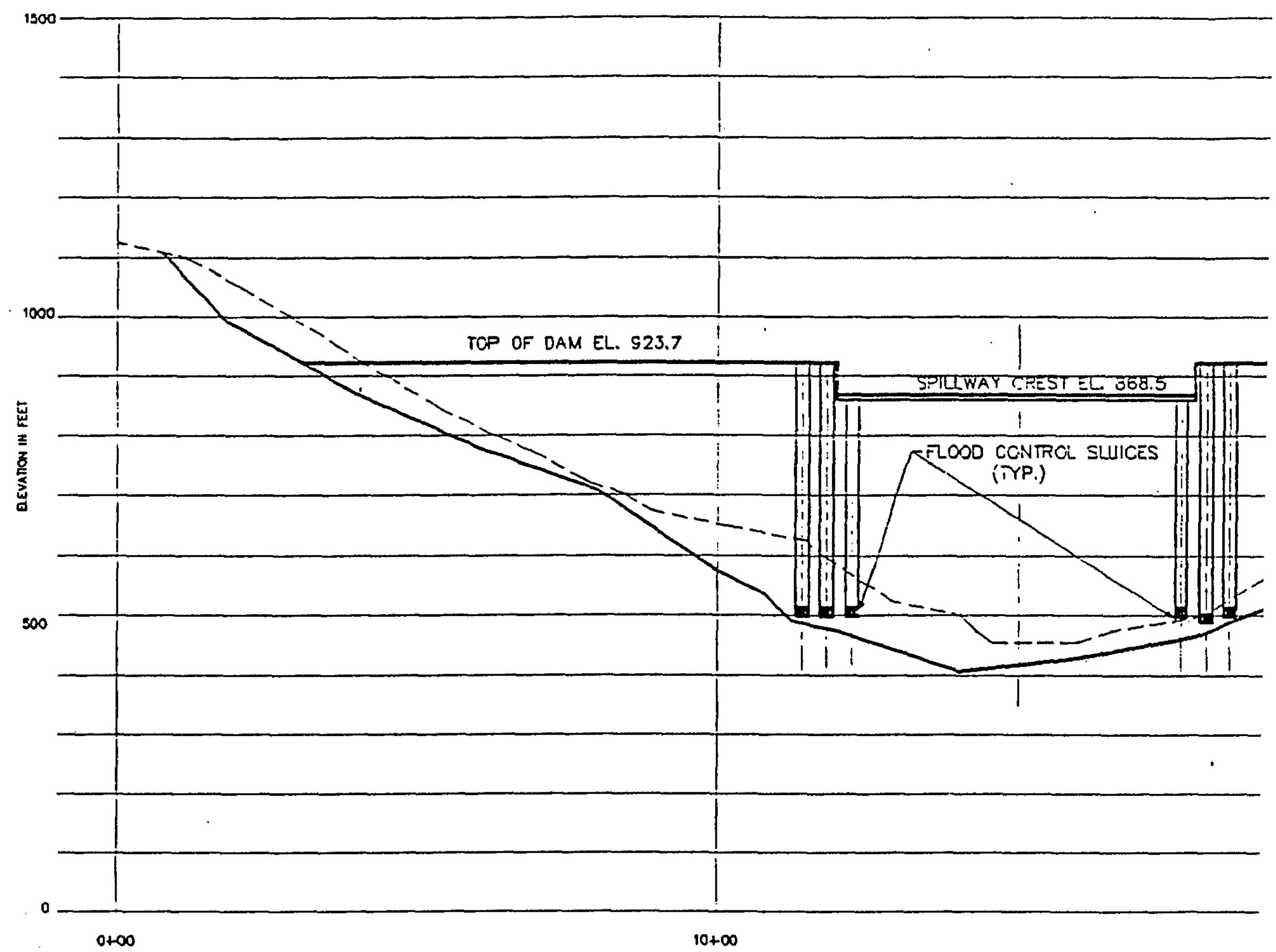


AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

SELECTED PLAN
DAM SITE RM. 20.1
SITE PLAN

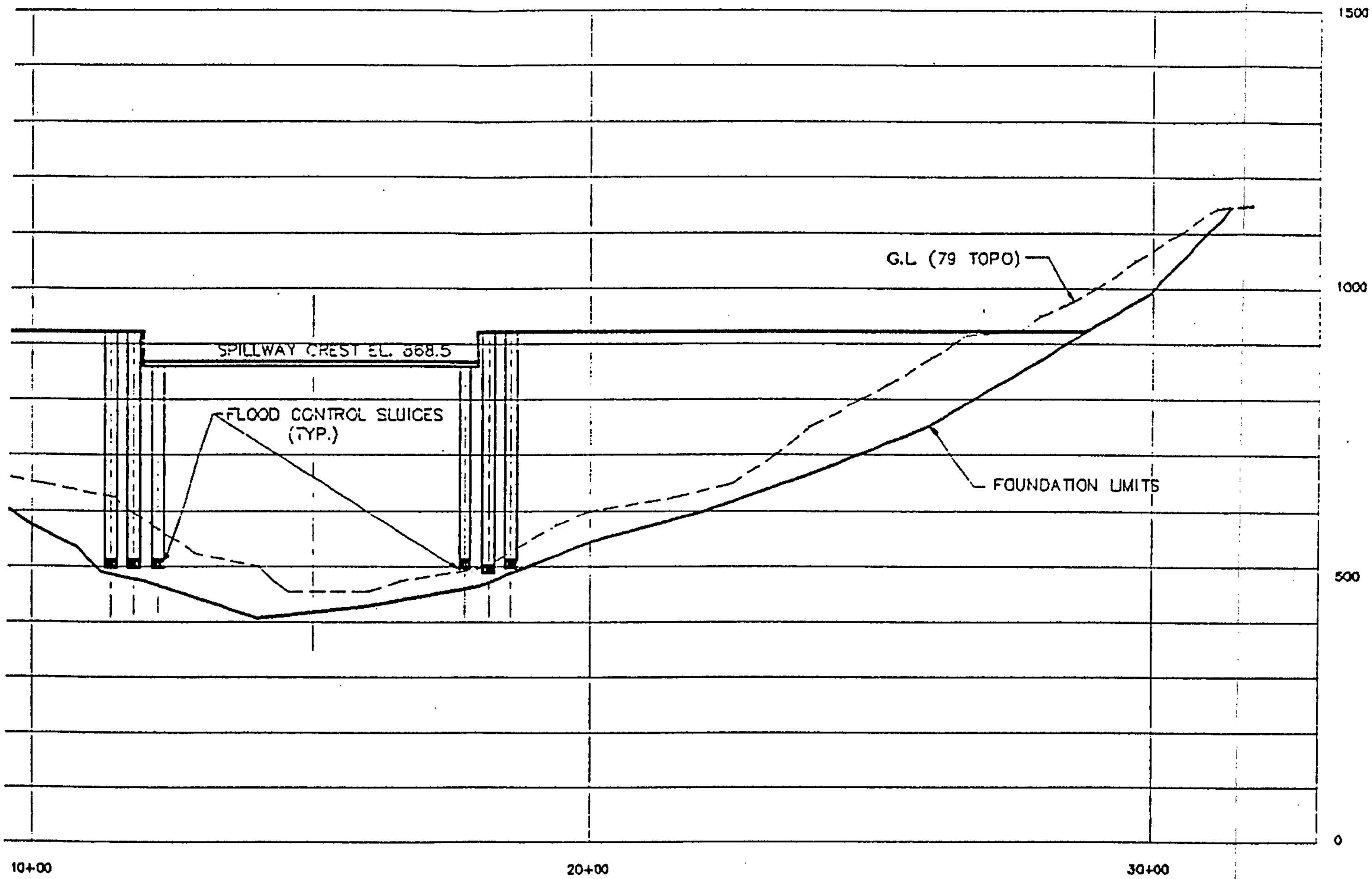
3

DESIGNED:	RLP		SACRAMENTO DISTRICT,
DRAWN:	GEB		CORPS OF ENGINEERS
CHECKED:	PLP	DATE:	SEPT. 1991



PROFILE OF UPSTREAM FAC

SCALE: 1"=300'



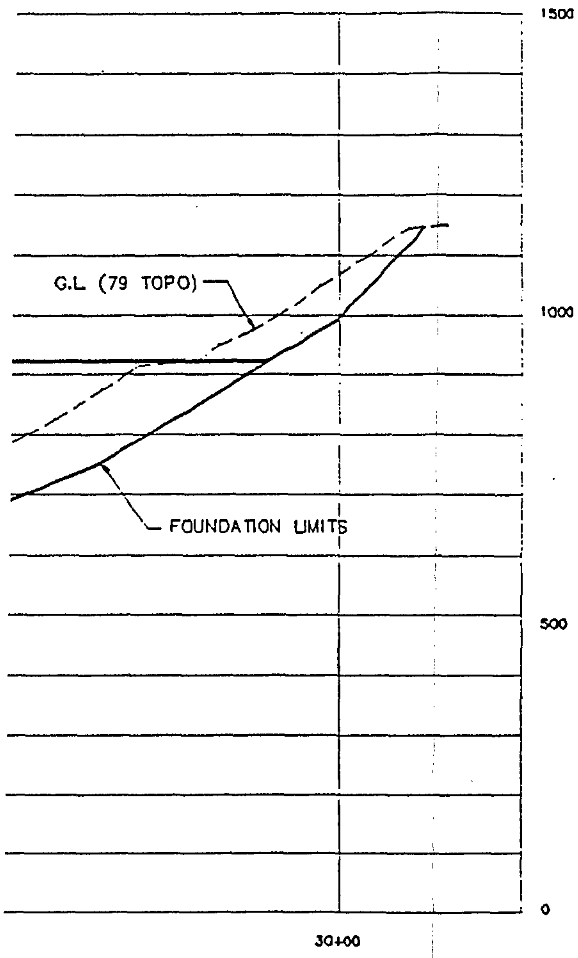
PROFILE OF UPSTREAM FACE

SCALE: 1"=300'

AMER

DESIGNED:
DRAWN:
CHECKED:

2



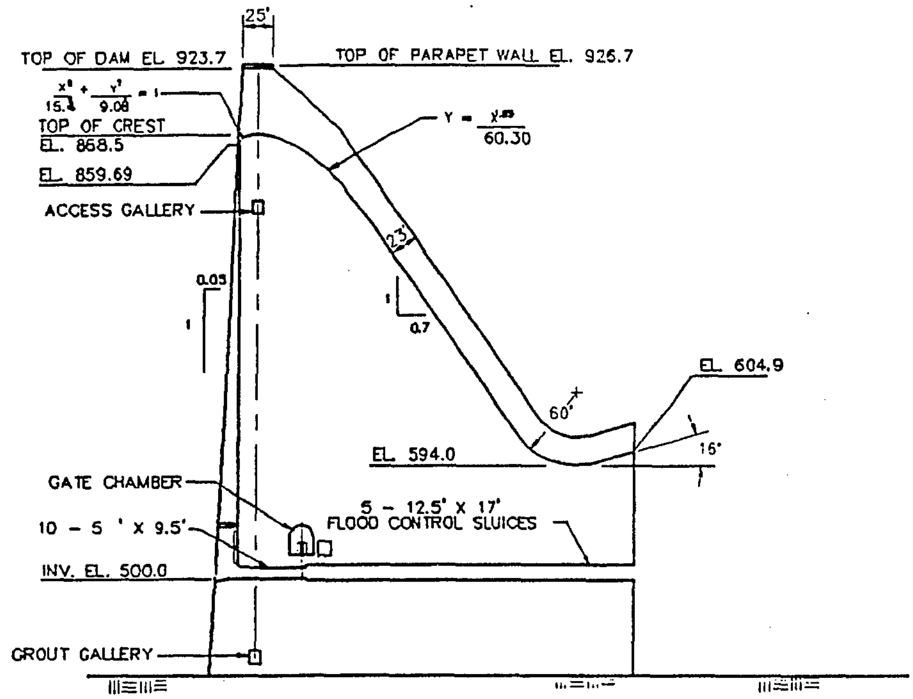
AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

FLOOD CONTROL DAM
SELECTED PLAN
PROFILE

3

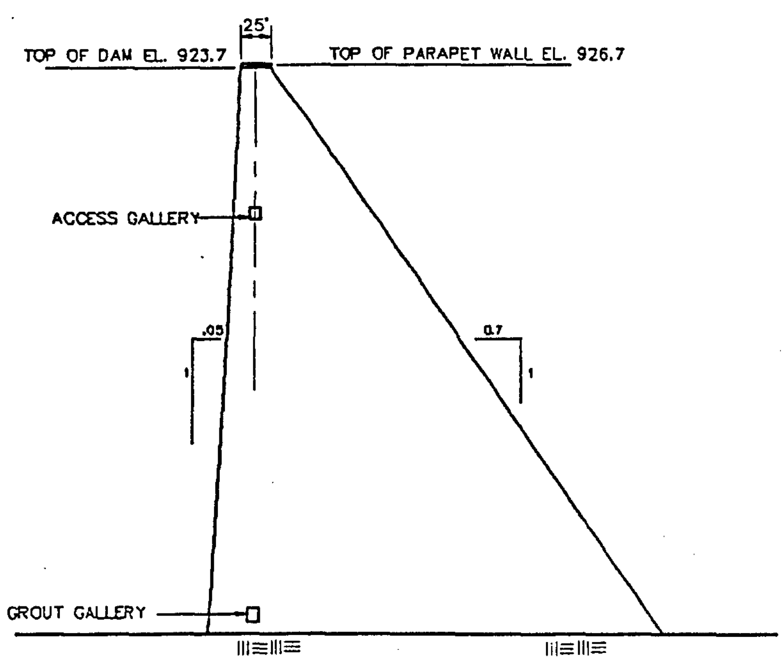
DESIGNED:	[illegible]		SACRAMENTO DISTRICT,
DRAWN:	[illegible]		CORPS OF ENGINEERS
CHECKED:	[illegible]	DATE:	NOV. 1991

PLATE 8



SPILLWAY SECTION

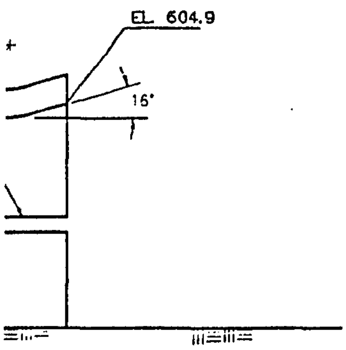
NTS

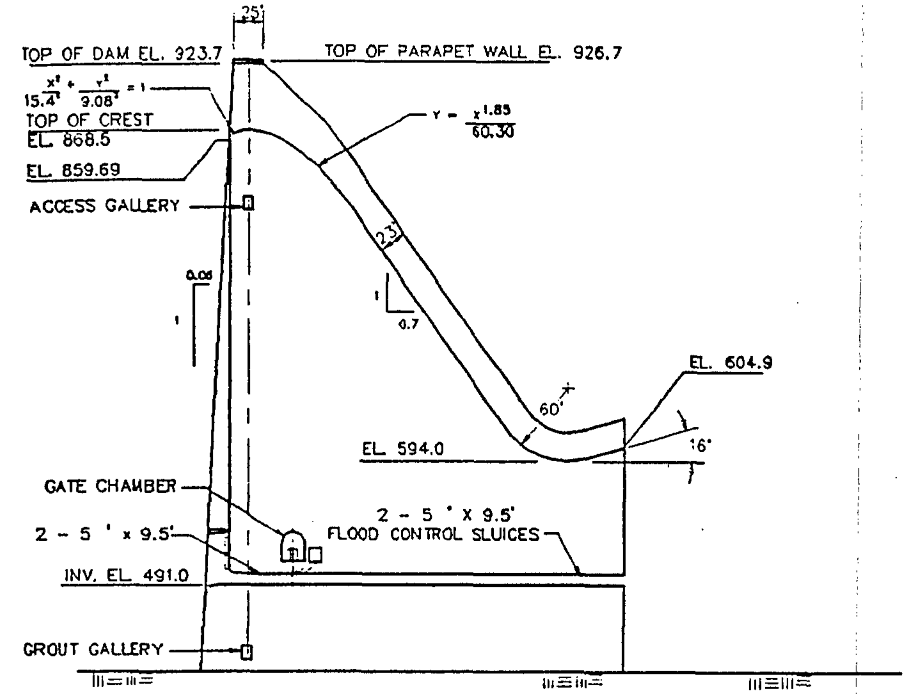


TYPICAL NON-OVERFLOW SECTION

NTS

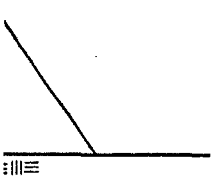
925.7





SPILLWAY SECTION
NTS

926.7



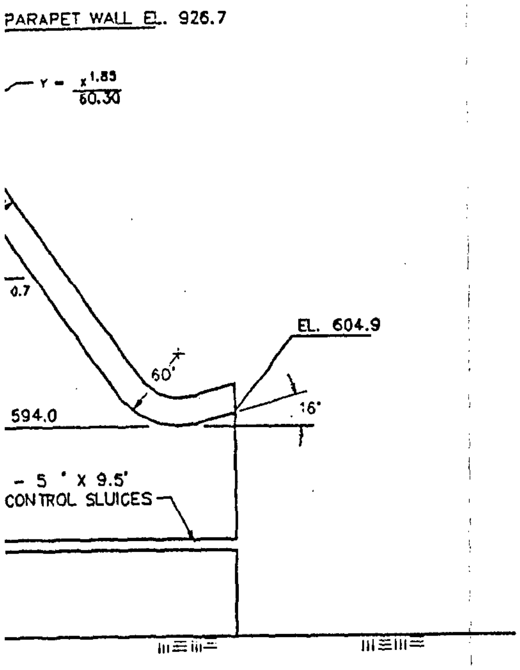
SECTION

AMERICAN RIVER

FLOC

SI

DESIGNED: RLP
DRAWN: GEB
CHECKED: RLP



AY SECTION
NTS

3

AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

FLOOD CONTROL DAM
SELECTED PLAN
SECTIONS

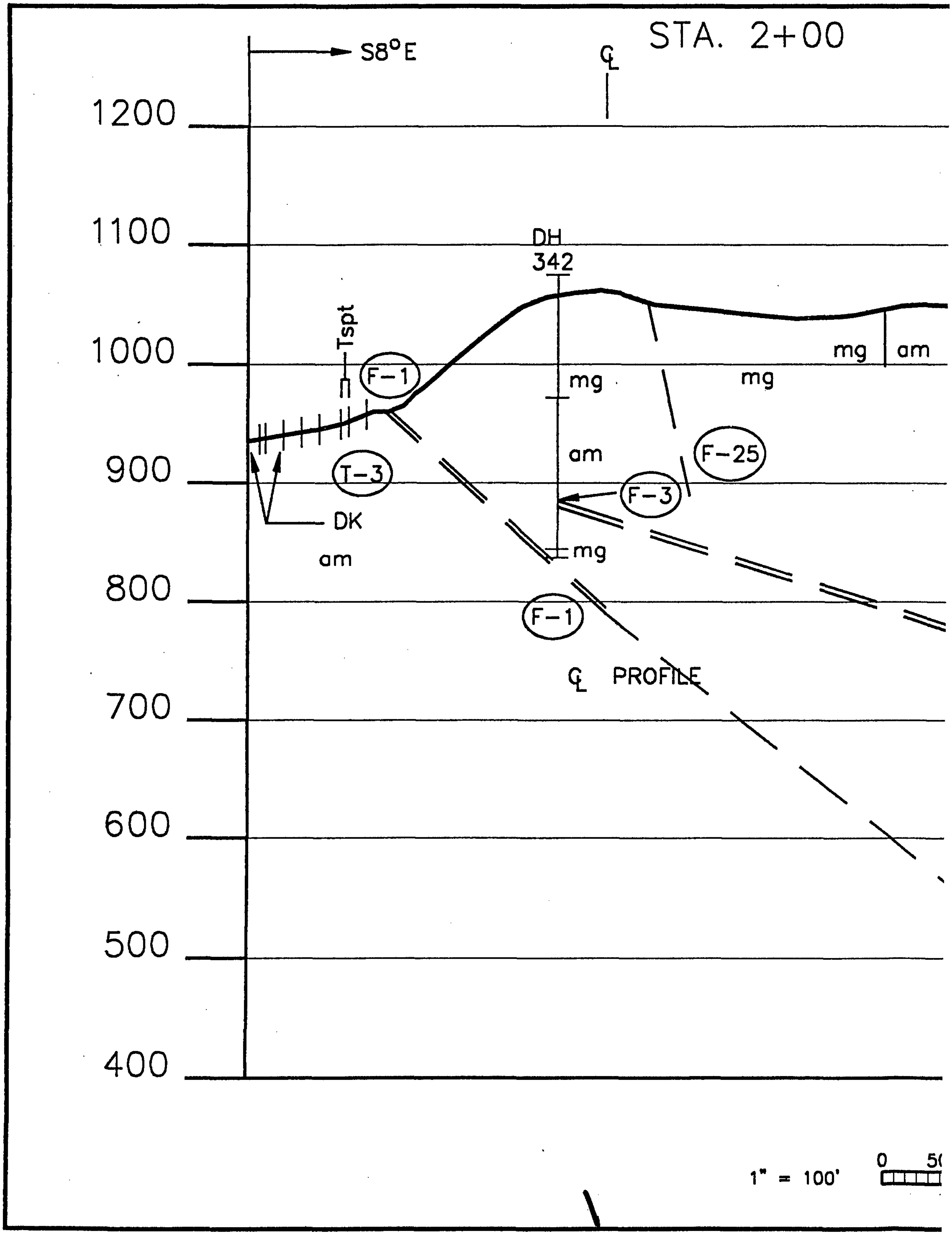
DESIGNED: RLP		SACRAMENTO DISTRICT,
DRAWN: GEB		CORPS OF ENGINEERS
CHECKED: RLP	DATE:	NOV. 1991

PLATE 9

NOTE: PLATE 10 IS TOO LARGE TO BE INCLUDED IN THIS APPENDIX
IT IS ON FILE IN THE SACRAMENTO DISTRICT
BLUE LINE COPIES HAVE BEEN SENT AS A SEPARATE PACKAGE TO COE REVIEWERS

NOTE: PLATE 11 IS TOO LARGE TO BE INCLUDED IN THIS APPENDIX
IT IS ON FILE IN THE SACRAMENTO DISTRICT
BLUE LINE COPIES HAVE BEEN SENT AS A SEPARATE PACKAGE TO COE REVIEWERS

NOTE: PLATE 12 IS TOO LARGE TO BE INCLUDED IN THIS APPENDIX
IT IS ON FILE IN THE SACRAMENTO DISTRICT
BLUE LINE COPIES HAVE BEEN SENT AS A SEPARATE PACKAGE TO COE REVIEWERS



STA. 2+00
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℄
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1100
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DH
342
Tspt
F-1
T-3
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am
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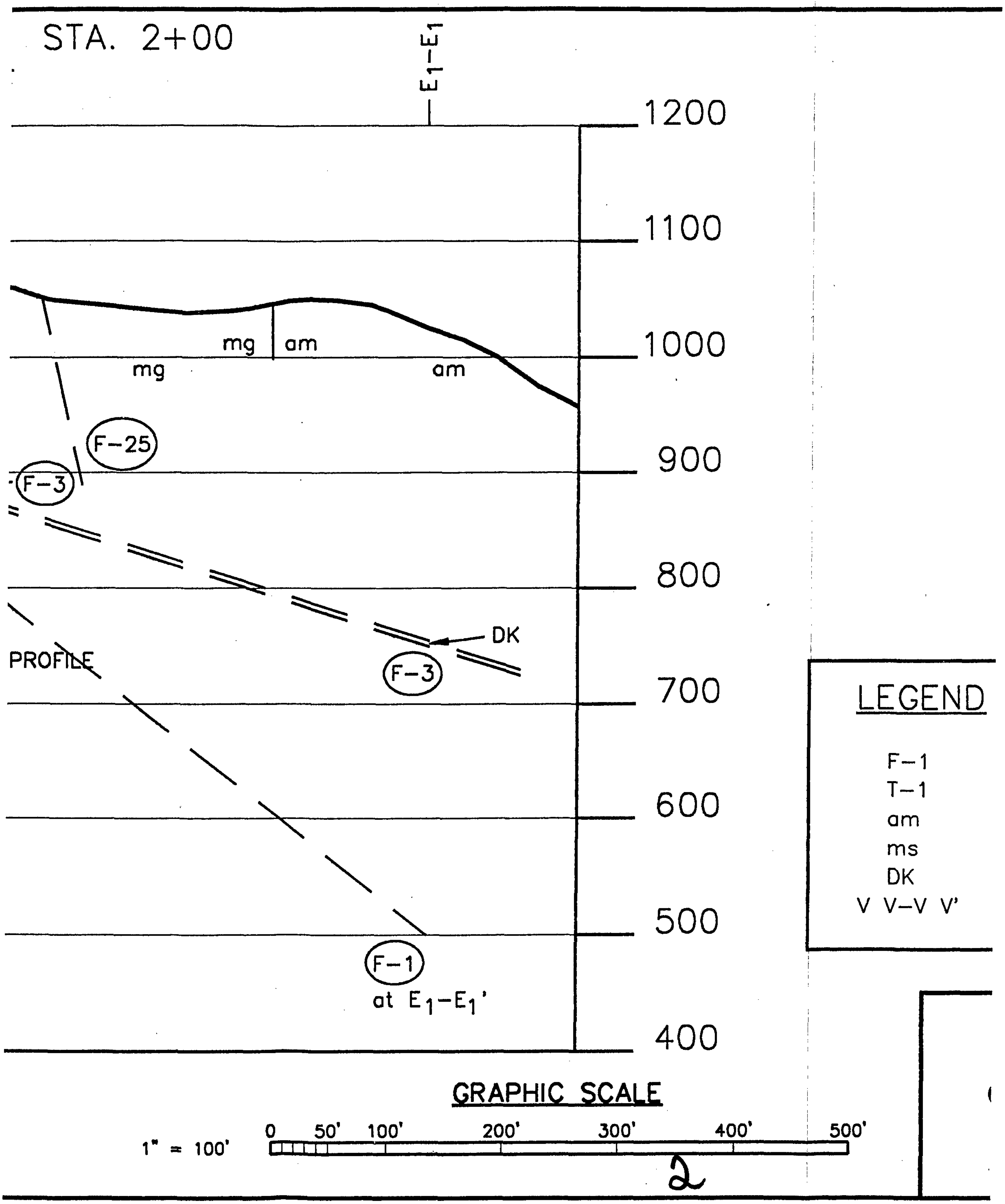
mg
F-3
F-25
F-1
mg
mg
am
℄ PROFILE
1" = 100'
0


STA. 2+00
E1-E1
1200
1100
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F-3
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F-3
PROFILE
F-1
at E1-E1'
LEGEND
F-1
T-1
am
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GRAPHIC SCALE
1" = 100'
0
50'
100'
200'
300'
400'
500'
2

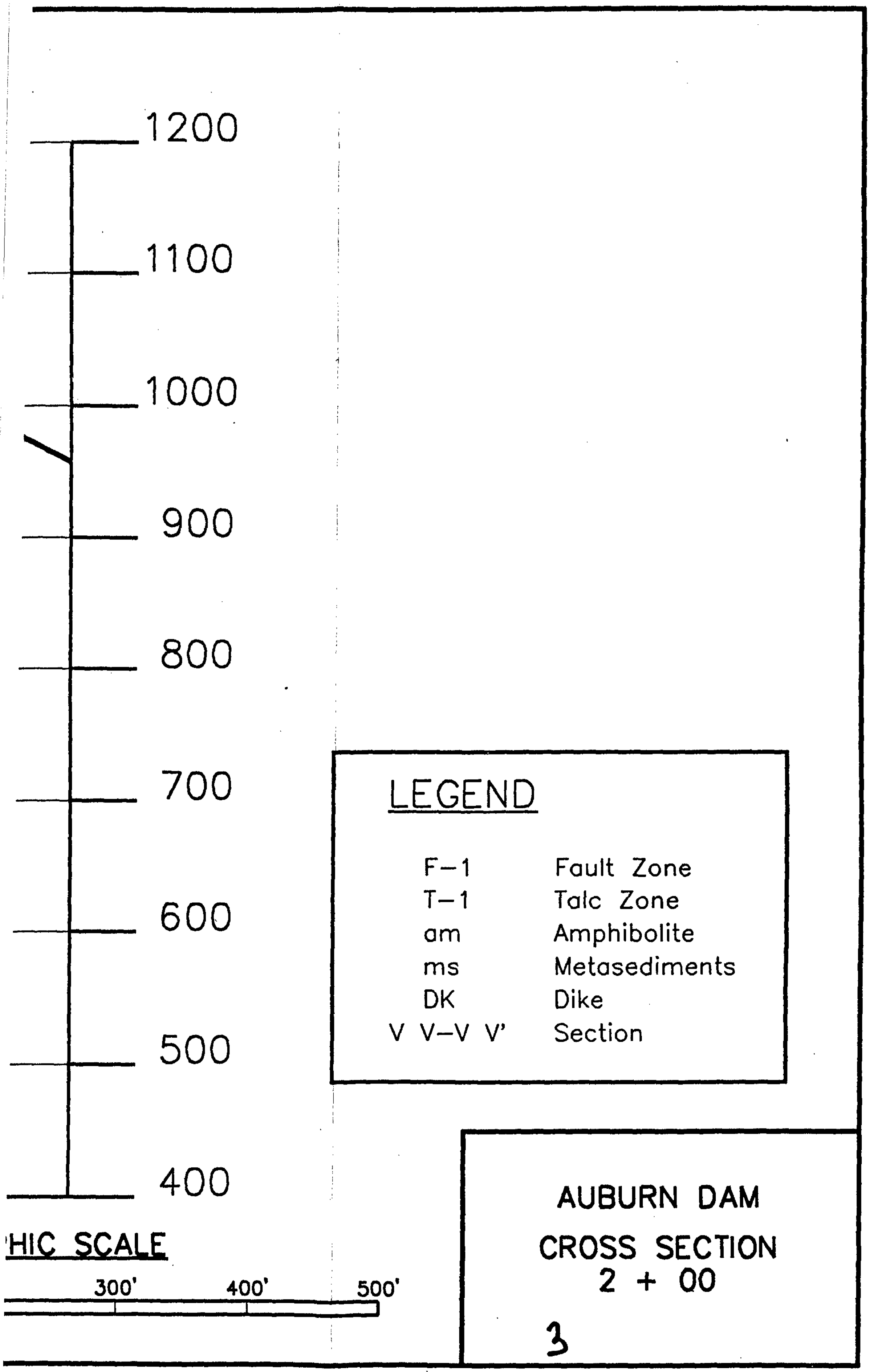
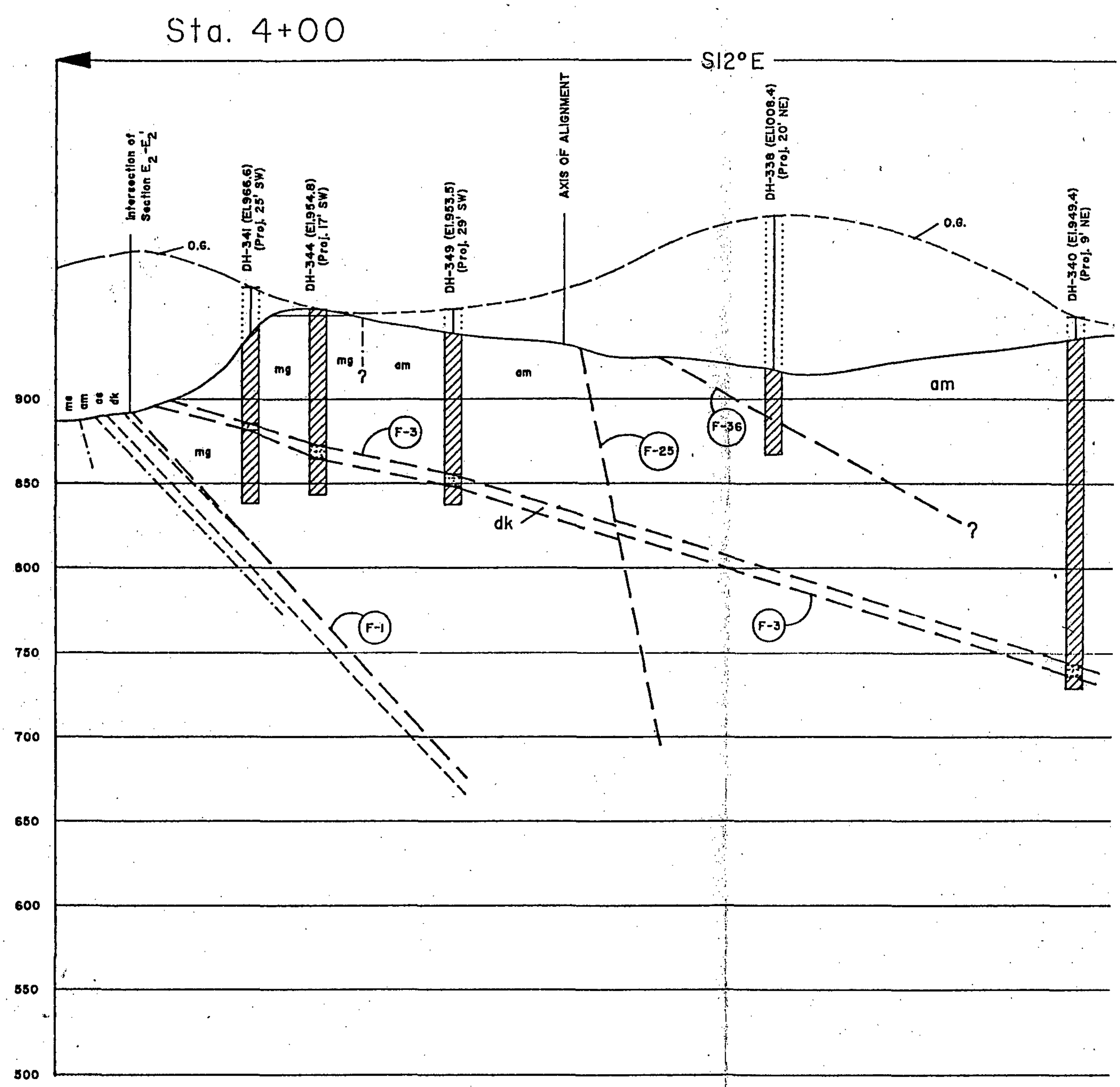
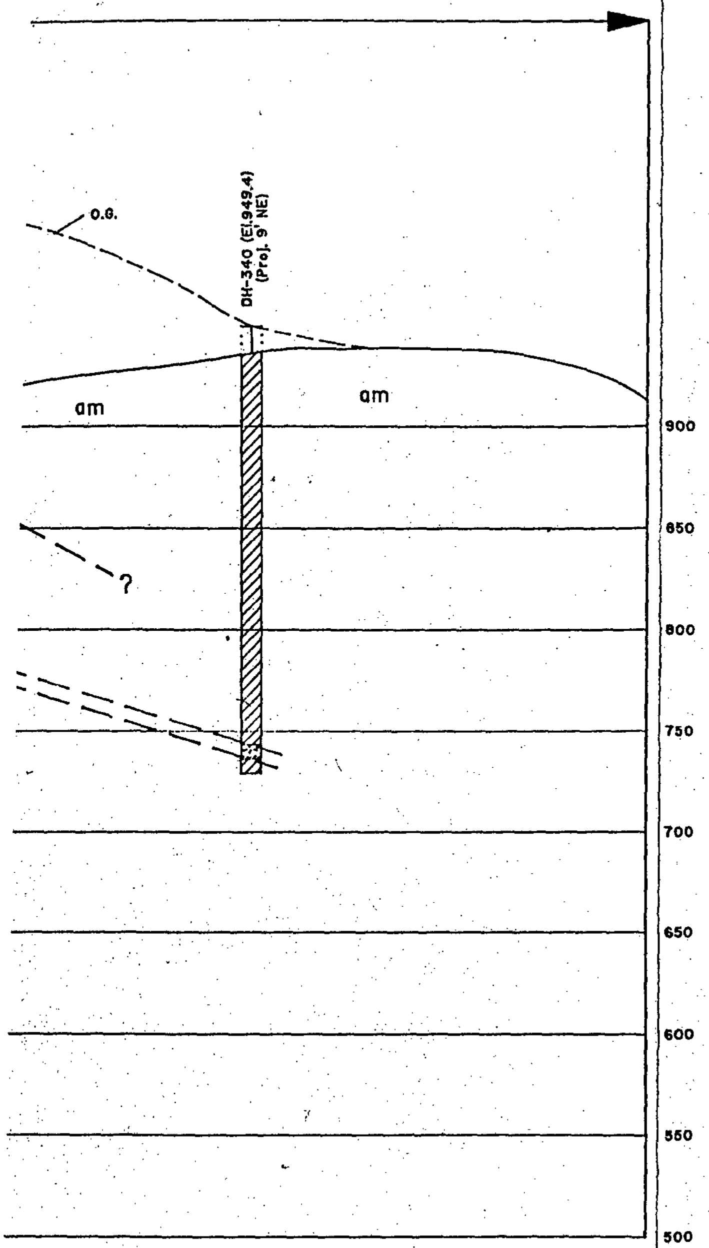


PLATE 13

ACAD FILE: \ACAD\LEILA-7\2
LATEST UPDATE: 11/15/91



Sta. 4+00
S12°E
Intersection of Section E_2-E_2'
DH-341 (El.966.6) (Proj. 25' SW)
DH-344 (El.954.8) (Proj. 17' SW)
DH-349 (El.953.5) (Proj. 29' SW)
AXIS OF ALIGNMENT
DH-338 (El.1008.4) (Proj. 20' NE)
DH-340 (El.949.4) (Proj. 9' NE)
O.G.
ms
am
as
dk
mg
am
F-3
F-25
F-36
F-1
900
850
800
750
700
650
600
550
500



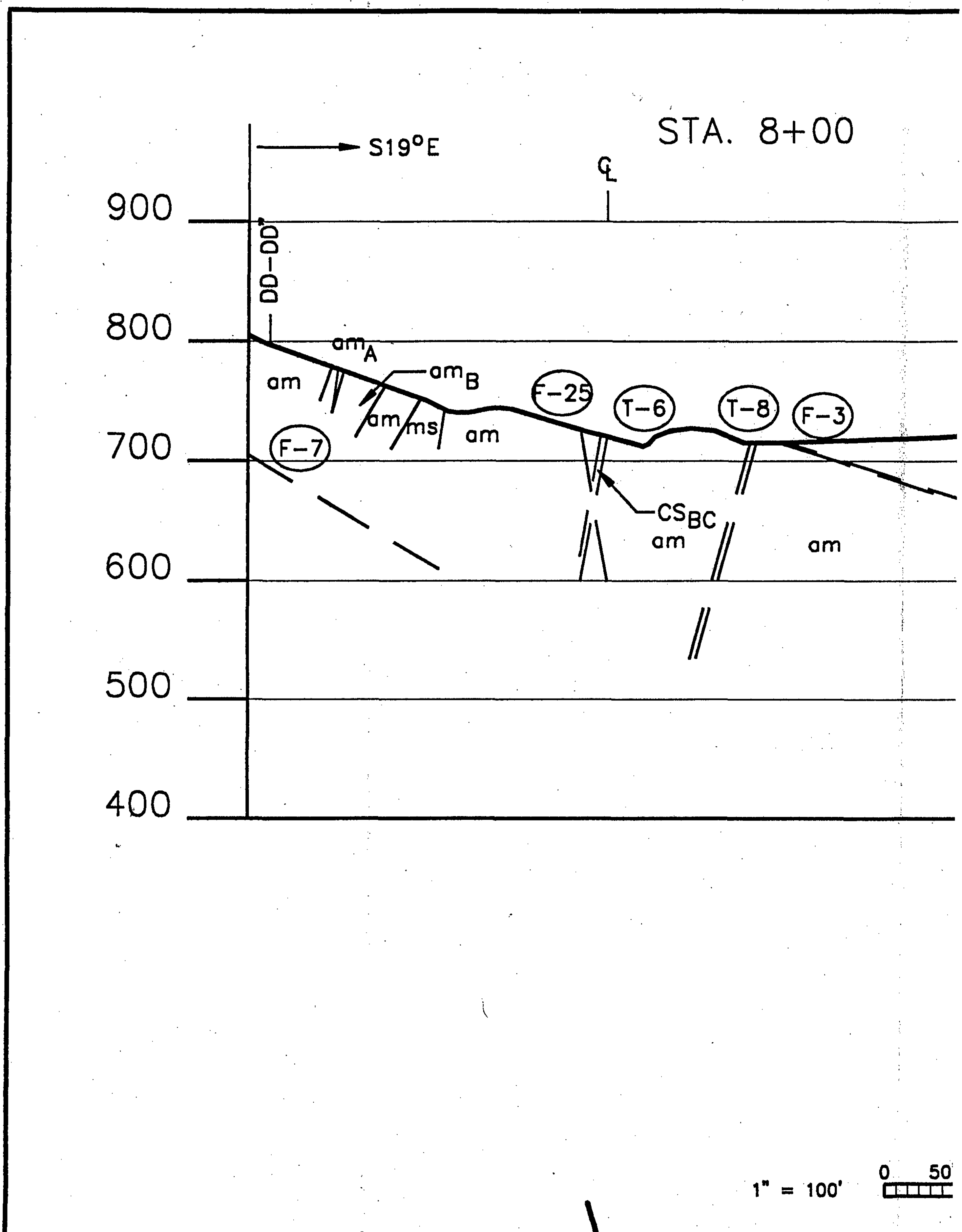
100' 0 100' 200'

AUBURN DAM

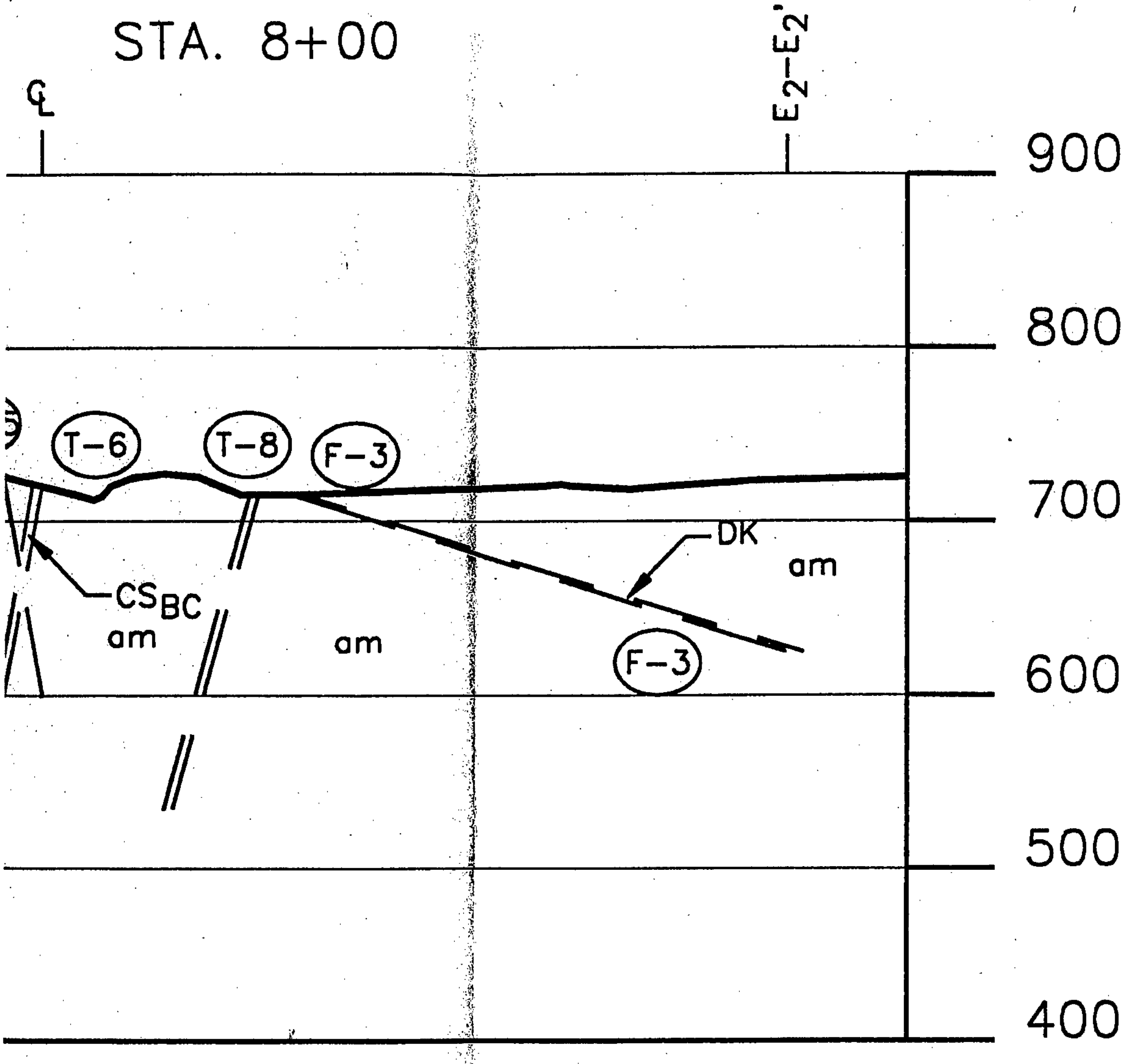
CROSS SECTION
4+00

2

PLATE 14



STA. 8+00
S19°E
℄
DD–DD'
900
800
700
600
500
400
am
amA
amB
am
ms
am
F-7
F-25
T-6
T-8
F-3
CSBC
am
am
1" = 100'
0
50



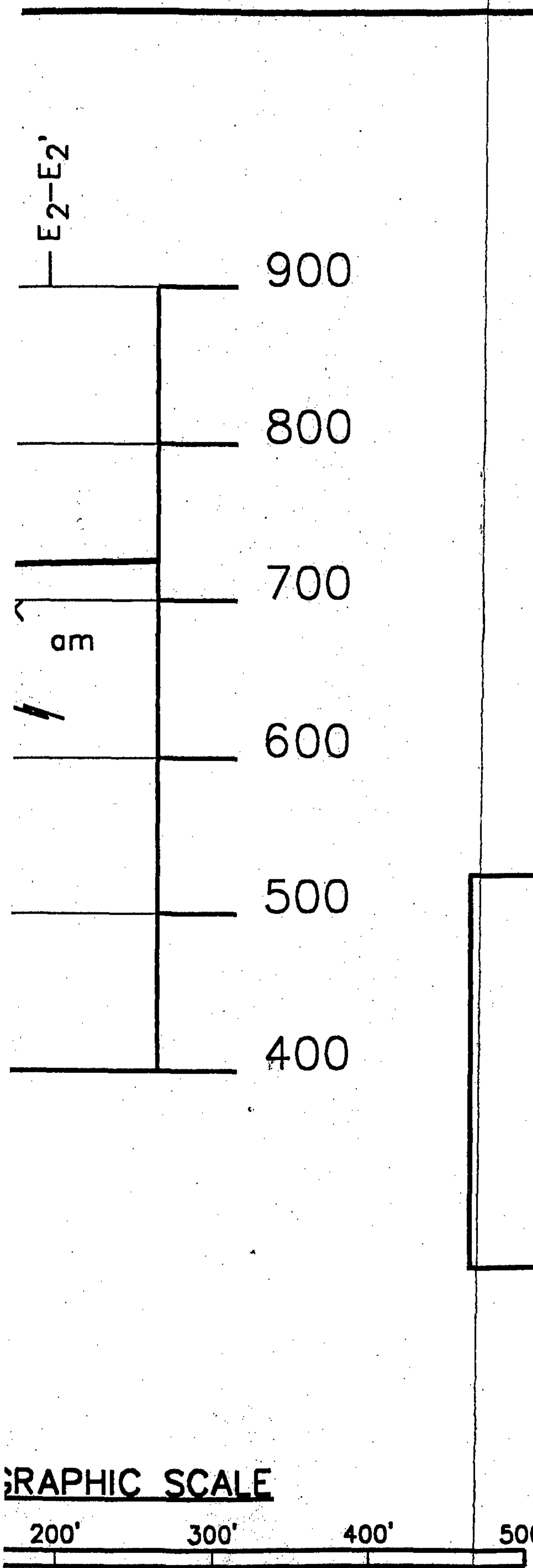
LEGEN

F-1
T-1
am
ms
DK
V V-V V'

GRAPHIC SCALE

1" = 100'   0   50'   100'   200'   300'   400'   500'

2



LEGEND

F-1	Fault Zone
T-1	Talc Zone
am	Amphibolite
ms	Metasediments
DK	Dike
V V-V V'	Section

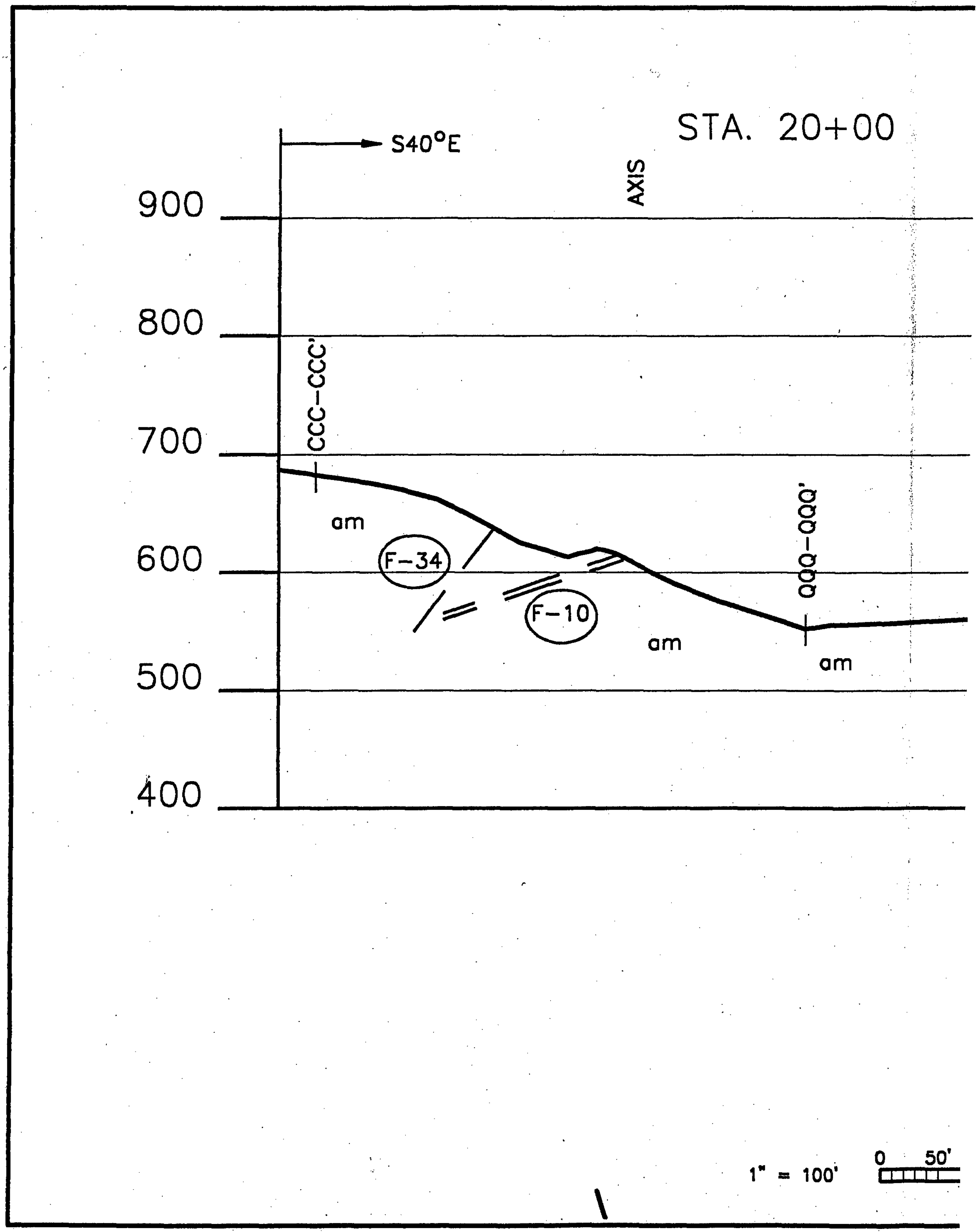
AUBURN DAM

CROSS SECTION
8 + 00

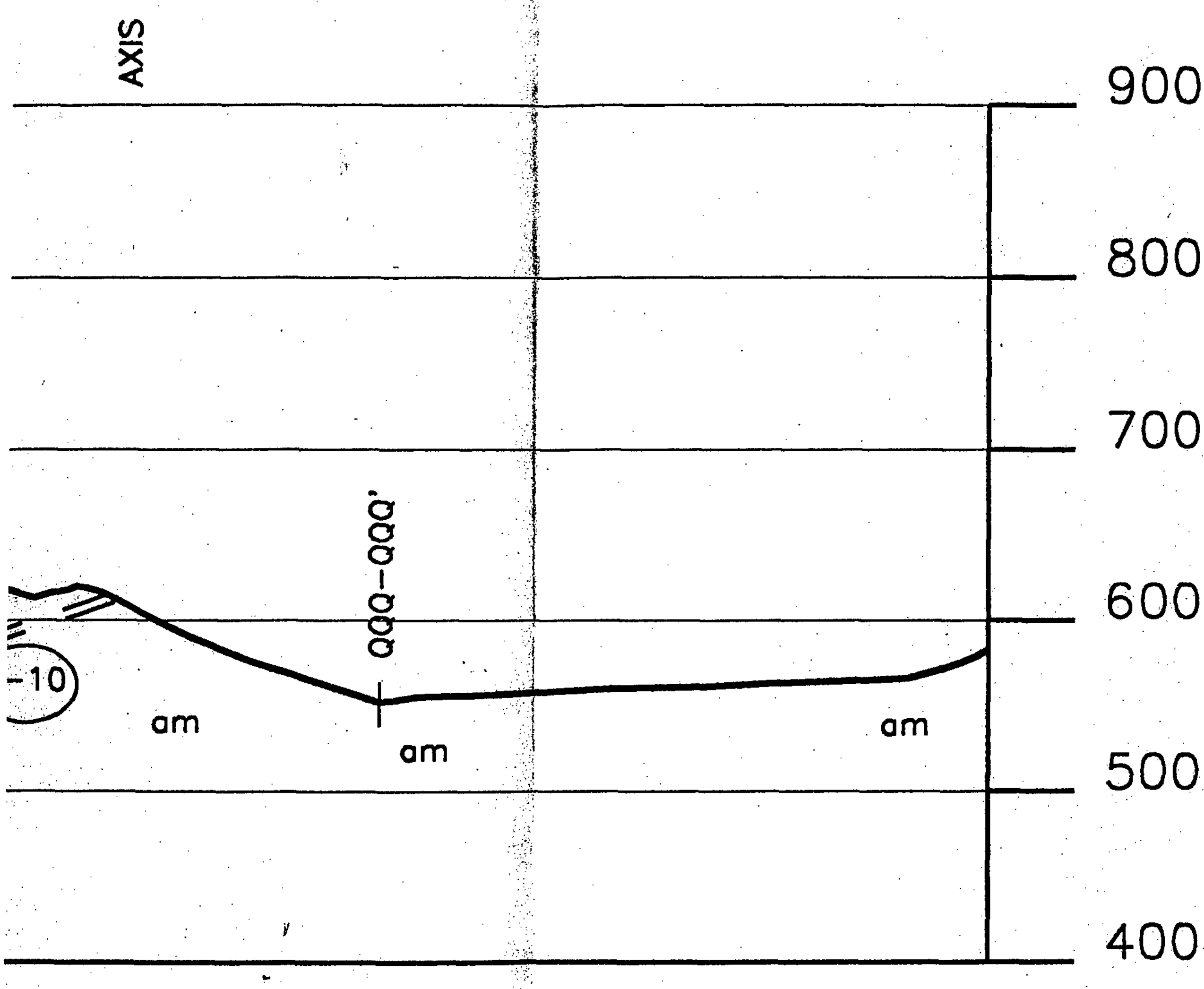
3

ACAD FILE: \ACAD\LEILA-7\2
LATEST UPDATE: 11/15/91

PLATE 15

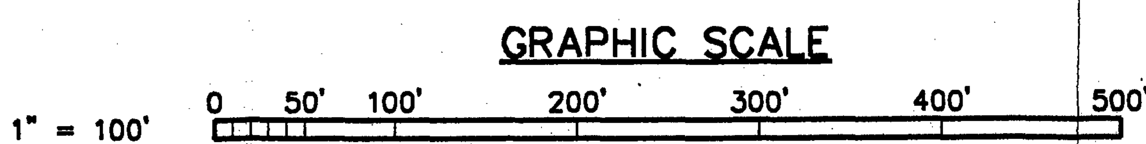

STA. 20+00
S40°E
AXIS
900
800
700
600
500
400
CCC-CCC'
QQQ-QQQ'
am
F-34
F-10
am
am
1" = 100'
0
50'

STA. 20+00

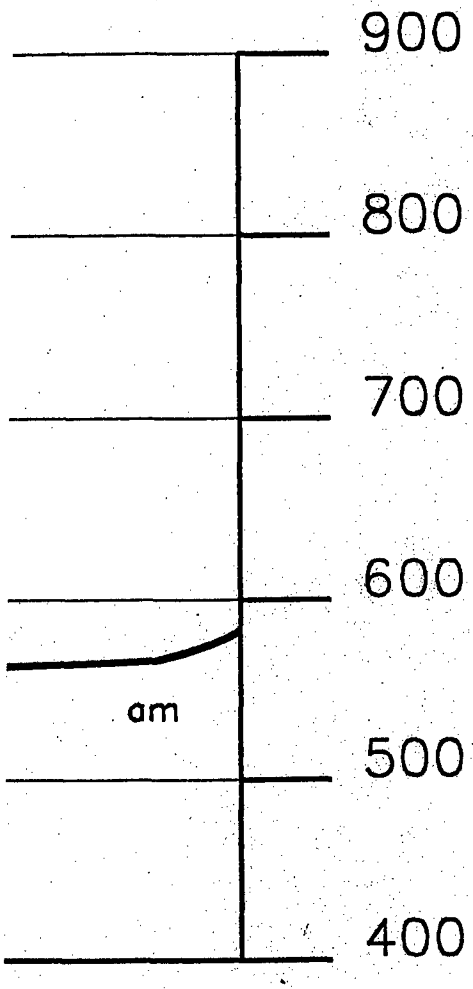


LEGE

F-1
T-1
am
ms
DK
V V-V



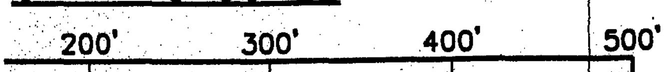
2



LEGEND

F−1	Fault Zone
T−1	Talc Zone
am	Amphibolite
ms	Metasediments
DK	Dike
V V−V V'	Section

GRAPHIC SCALE



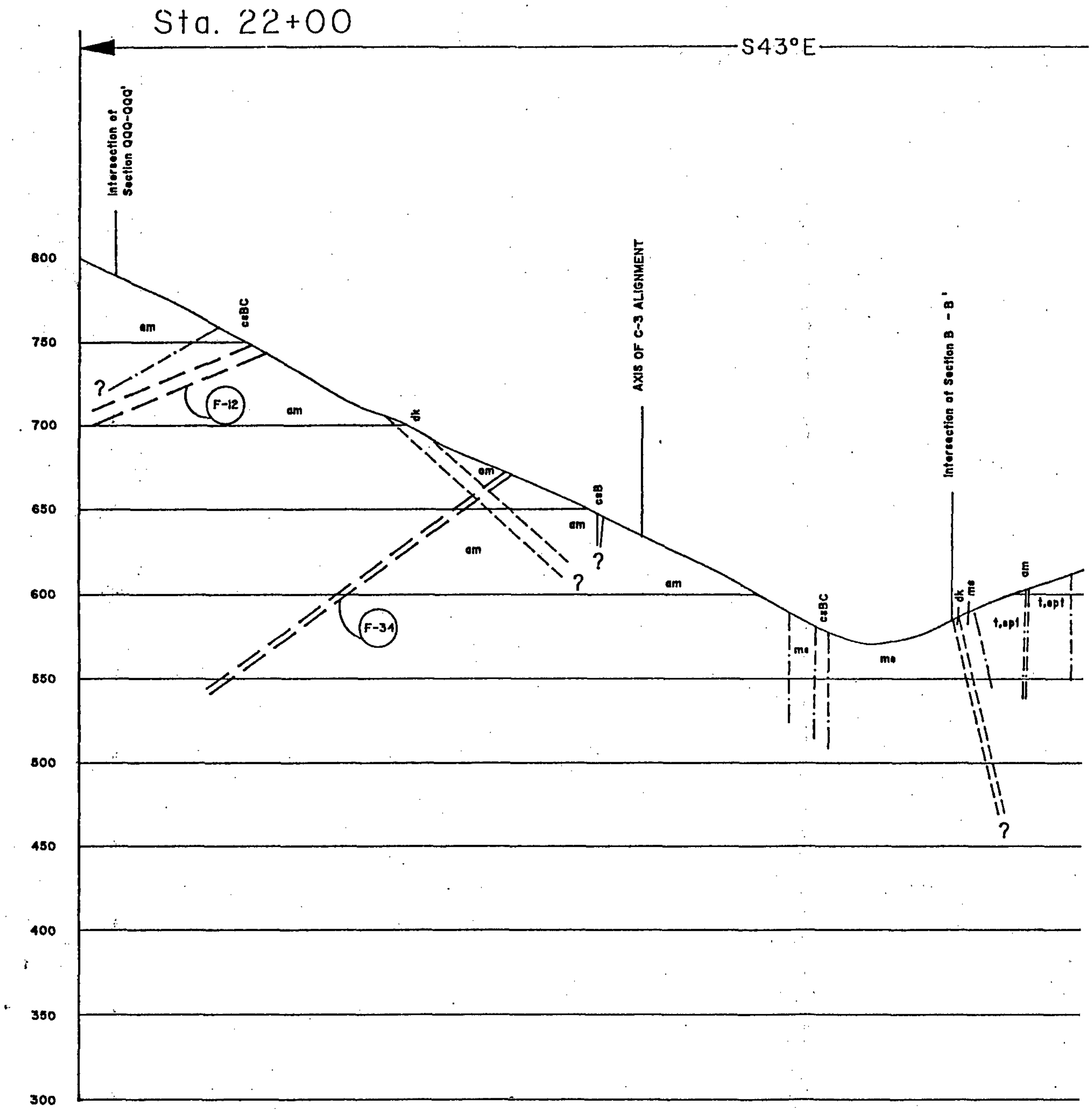
AUBURN DAM

CROSS SECTION
20 + 00

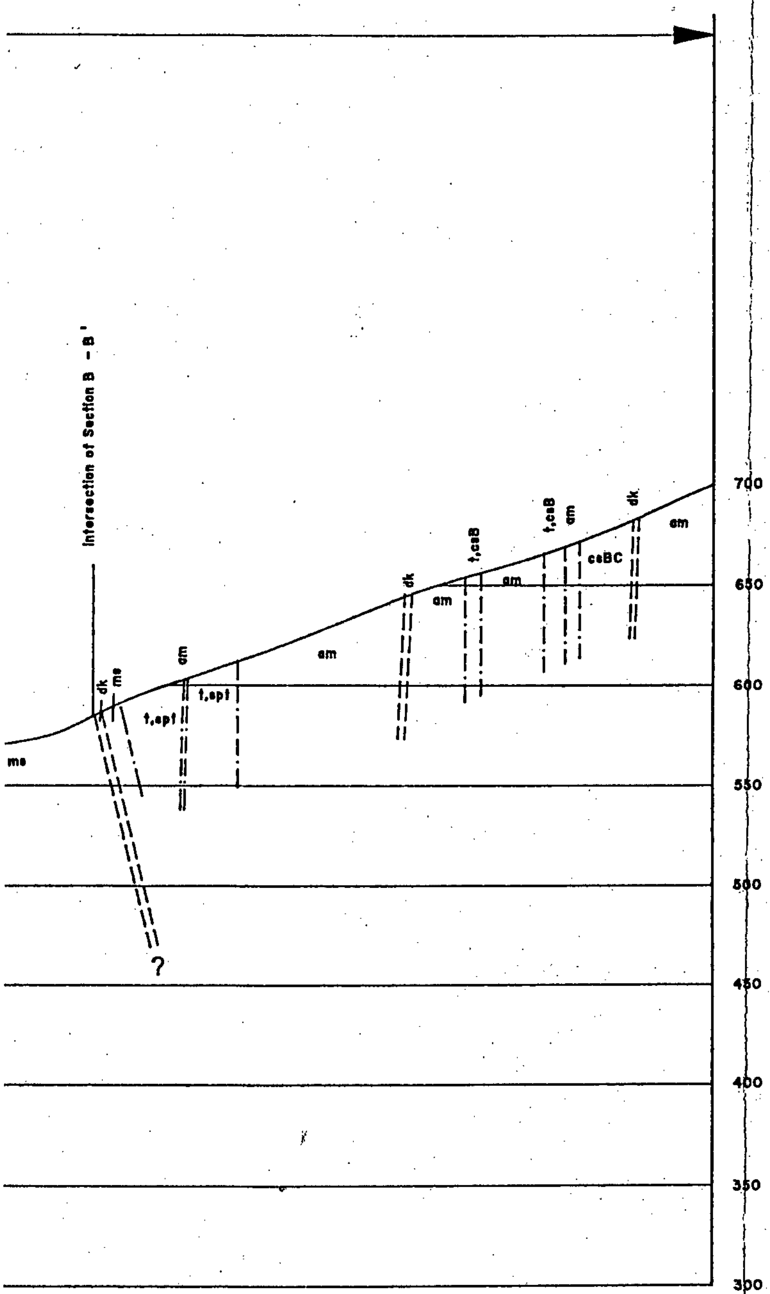
3

ACAD FILE: \ACAD\LEILA-7\2
LATEST UPDATE: 11/15/91

PLATE 16



Sta. 22+00
S43°E
Intersection of Section QQQ-QQQ'
AXIS OF C-3 ALIGNMENT
Intersection of Section B - B'
800
750
700
650
600
550
500
450
400
350
300
am
csBC
F-12
dk
csB
F-34
ms
t,spt
?



100' 0 100' 200'

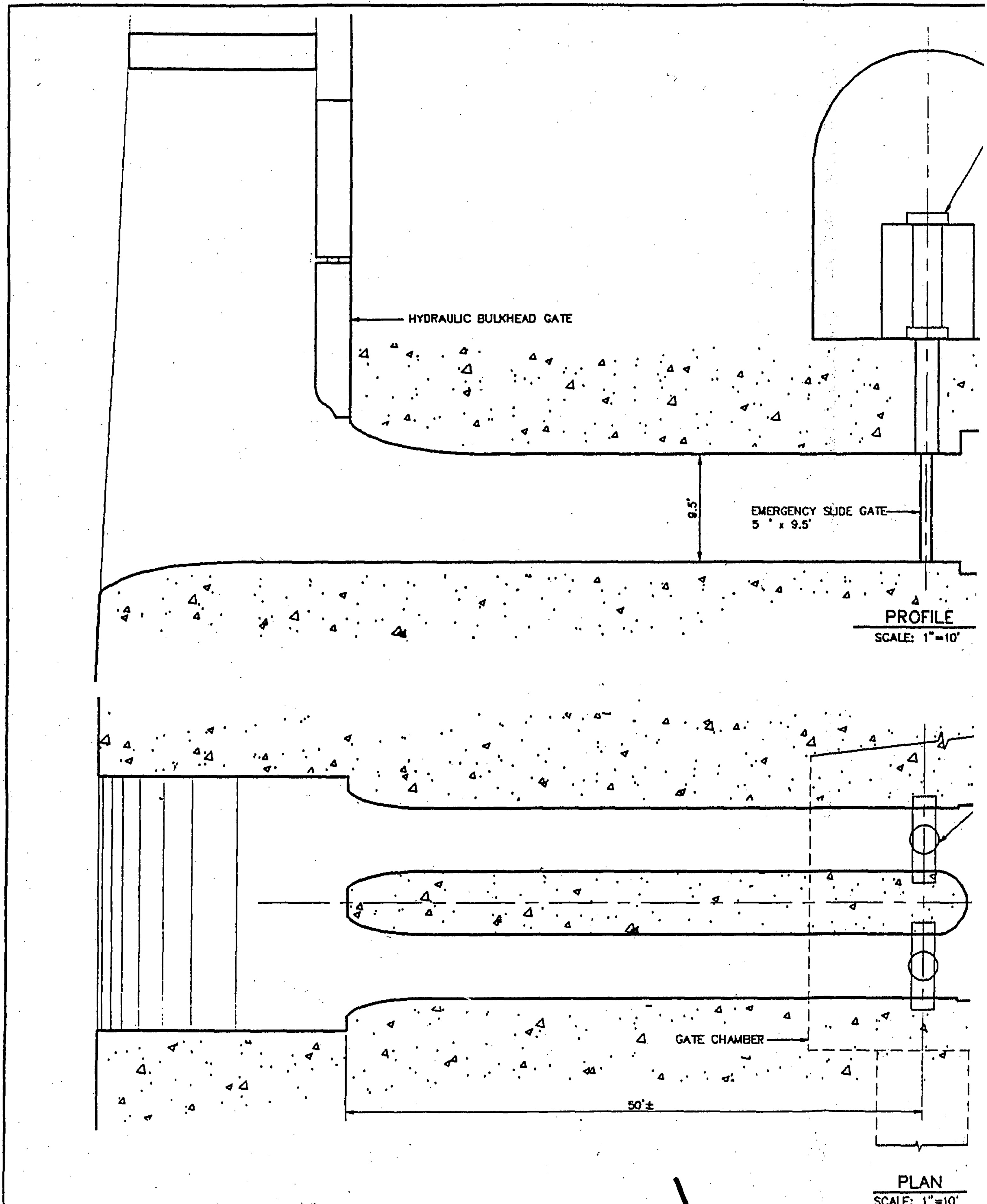
AUBURN DAM

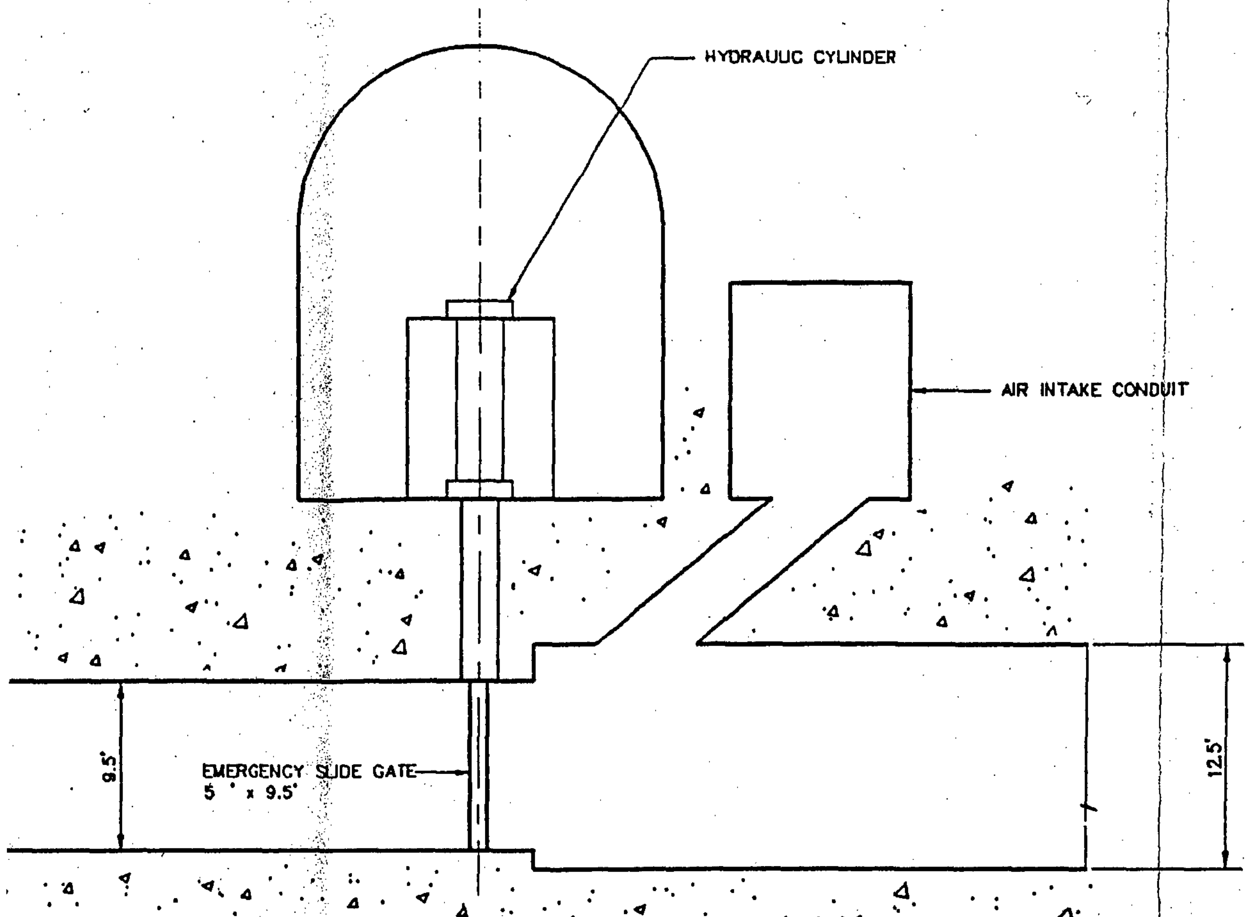
CROSS SECTION
22+00

2

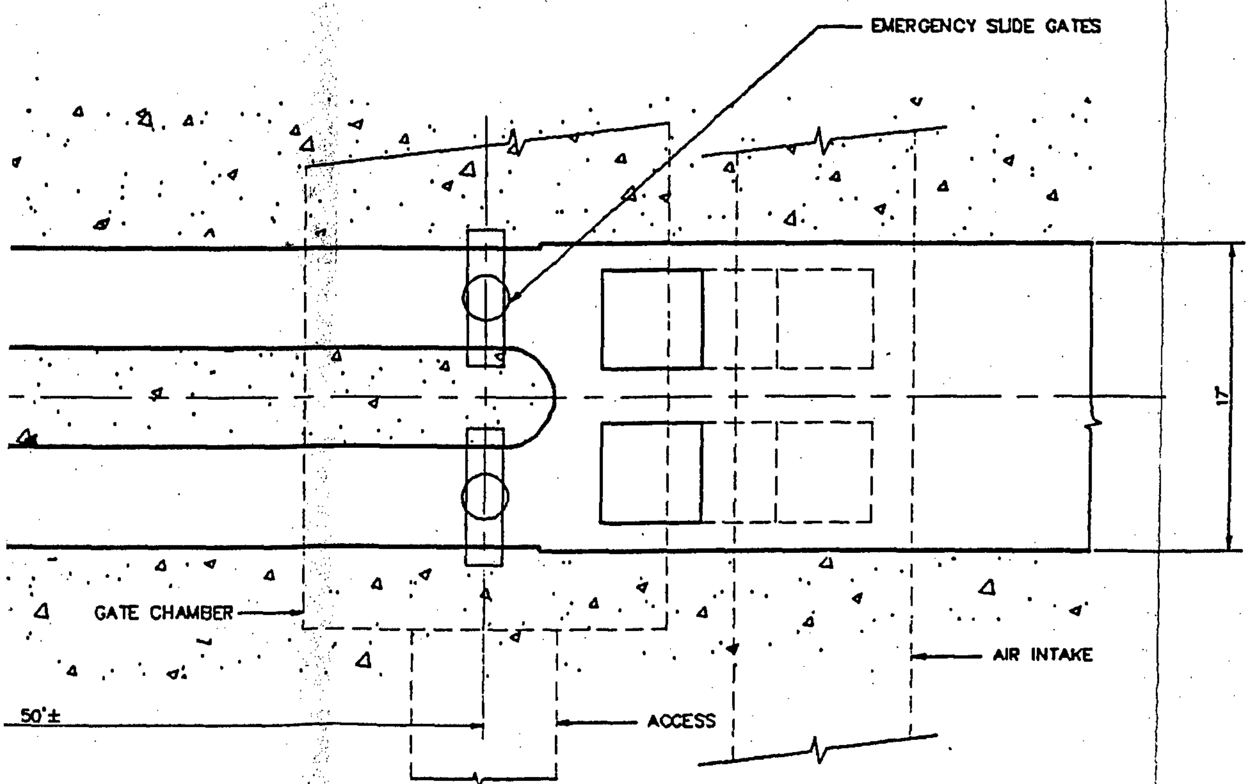
PLATE 17

NOTE: PLATES 18-30 ARE TOO LARGE TO BE INCLUDED IN THIS APPENDIX
IT IS ON FILE IN THE SACRAMENTO DISTRICT
BLUE LINE COPIES HAVE BEEN SENT AS A SEPARATE PACKAGE TO COE REVIEWERS


HYDRAULIC BULKHEAD GATE
9.5'
EMERGENCY SLIDE GATE
5 ' x 9.5'
PROFILE
SCALE: 1"=10'
GATE CHAMBER
50'±
PLAN
SCALE: 1"=10'



PROFILE
SCALE: 1"=10'



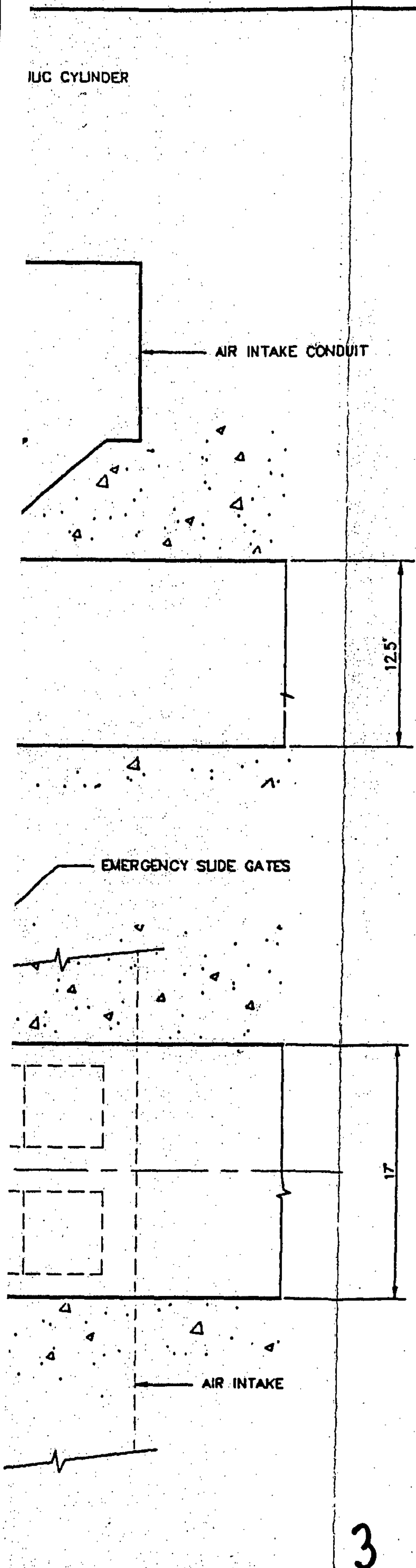
PLAN
SCALE: 1"=10'

2

AMERICAN RIVER WATERSH

FLOOD CC
SELEC
EMERGEN
PLAN ANI

DESIGNED: RLP
DRAWN: GEB
CHECKED: RLP
DATE



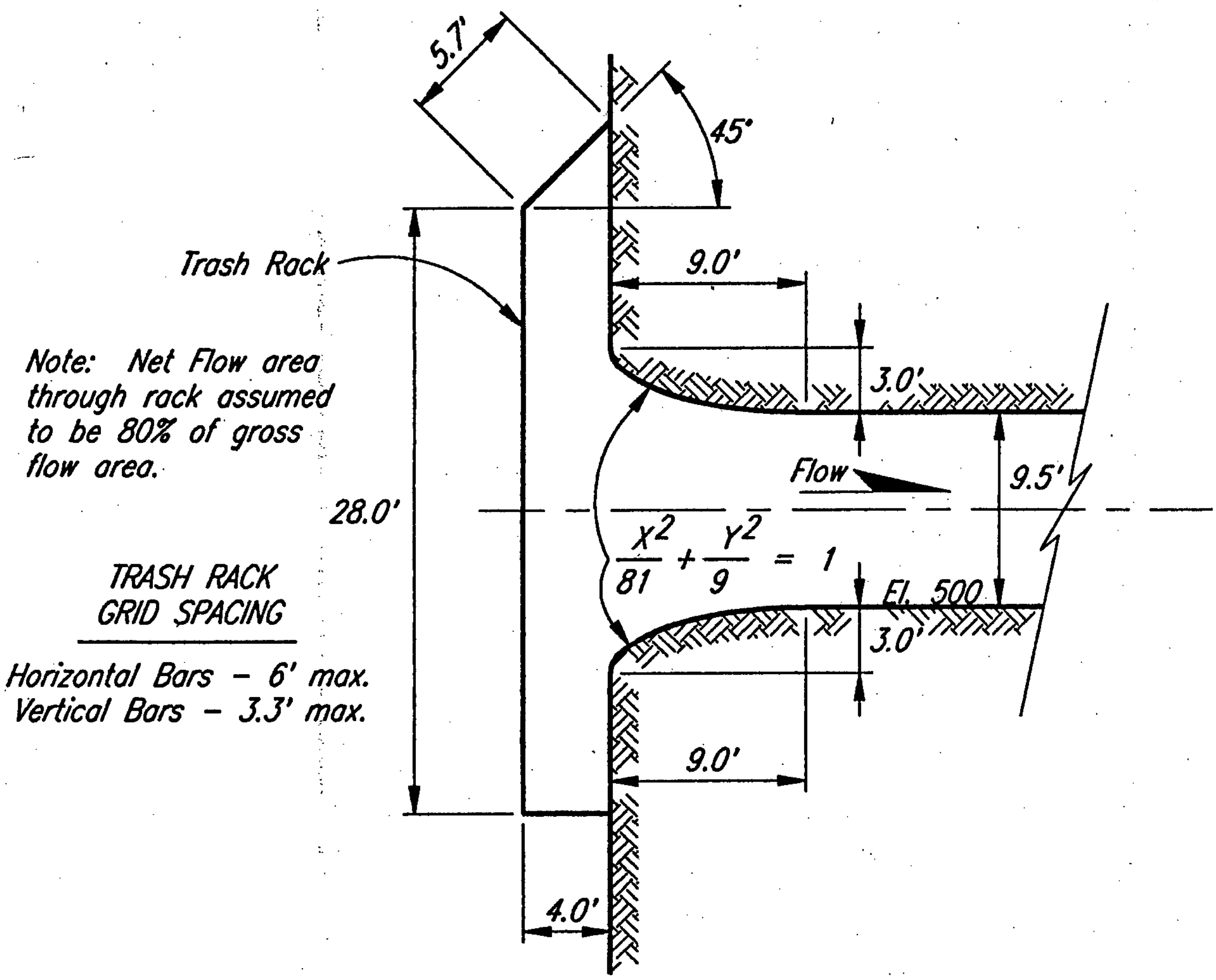
AMERICAN RIVER WATERSHED INVESTIGATION, CALIFORNIA

FLOOD CONTROL DAM
SELECTED PLAN
EMERGENCY GATES
PLAN AND PROFILE

DESIGNED: RLP
DRAWN: GEB
CHECKED: RLP DATE: NOV. 1991

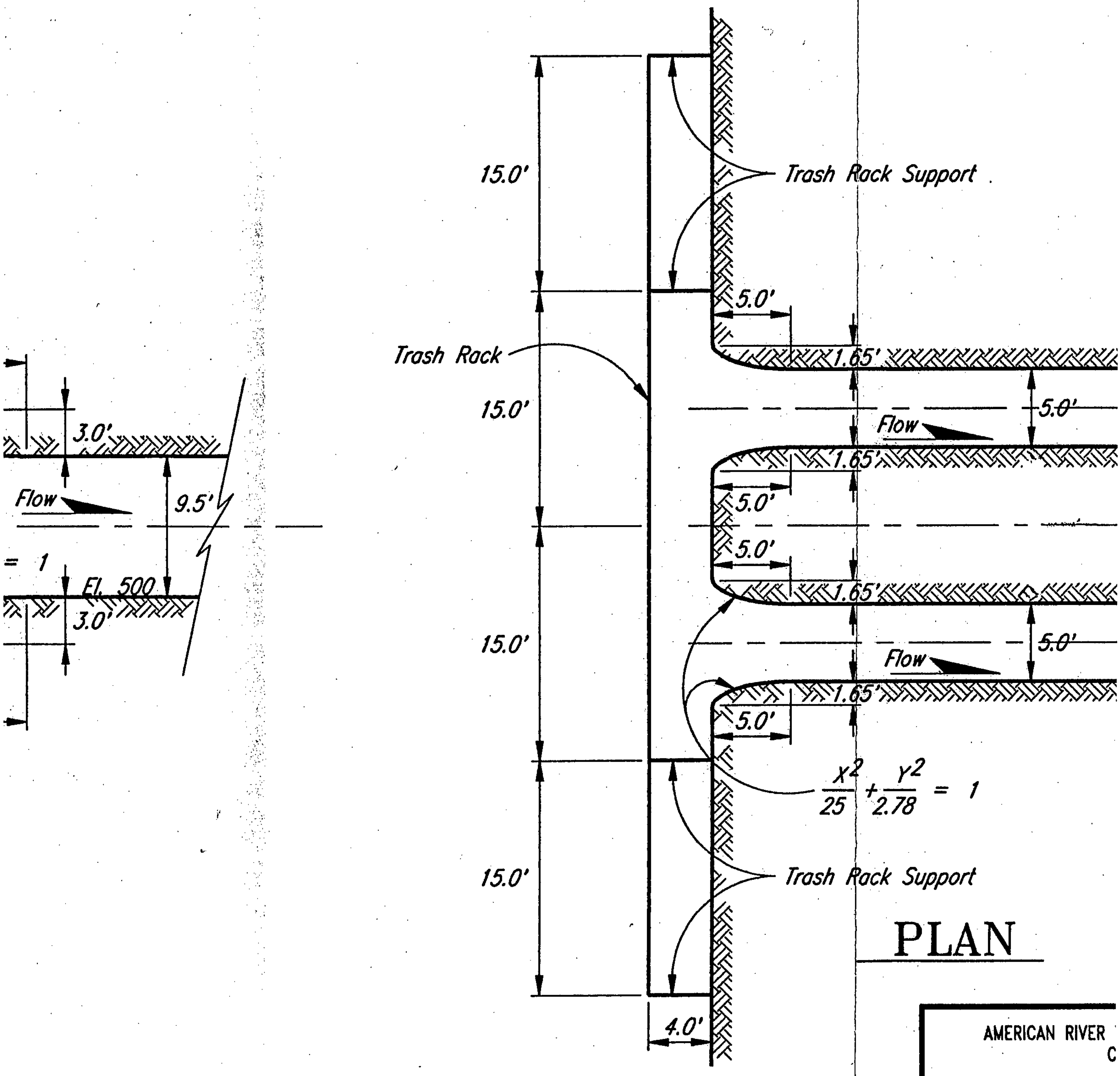
SACRAMENTO DISTRICT,
CORPS OF ENGINEERS

PLATE 31



5.7'
45°
Trash Rack
9.0'
Note: Net Flow area through rack assumed to be 80% of gross flow area.
3.0'
Flow
9.5'
28.0'
$\frac{X^2}{81} + \frac{Y^2}{9} = 1$
TRASH RACK GRID SPACING
Horizontal Bars – 6' max.
Vertical Bars – 3.3' max.
El. 500
3.0'
9.0'
4.0'

PROFILE



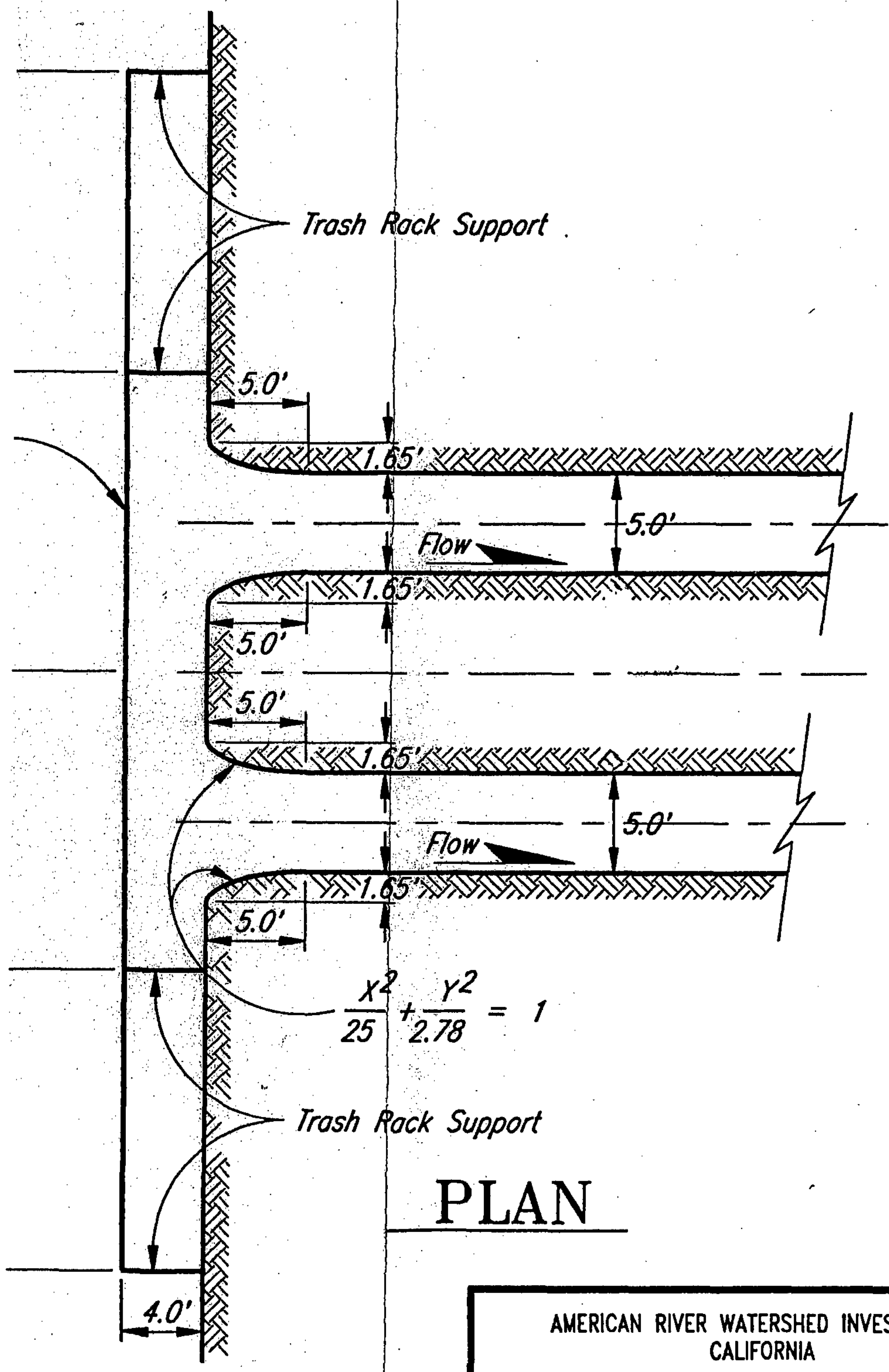
PLAN

AMERICAN RIVER
C

SELE
RM
FLOOD C
ENTRANCE AND T

SACRAMENTO DISTI
NOV

2



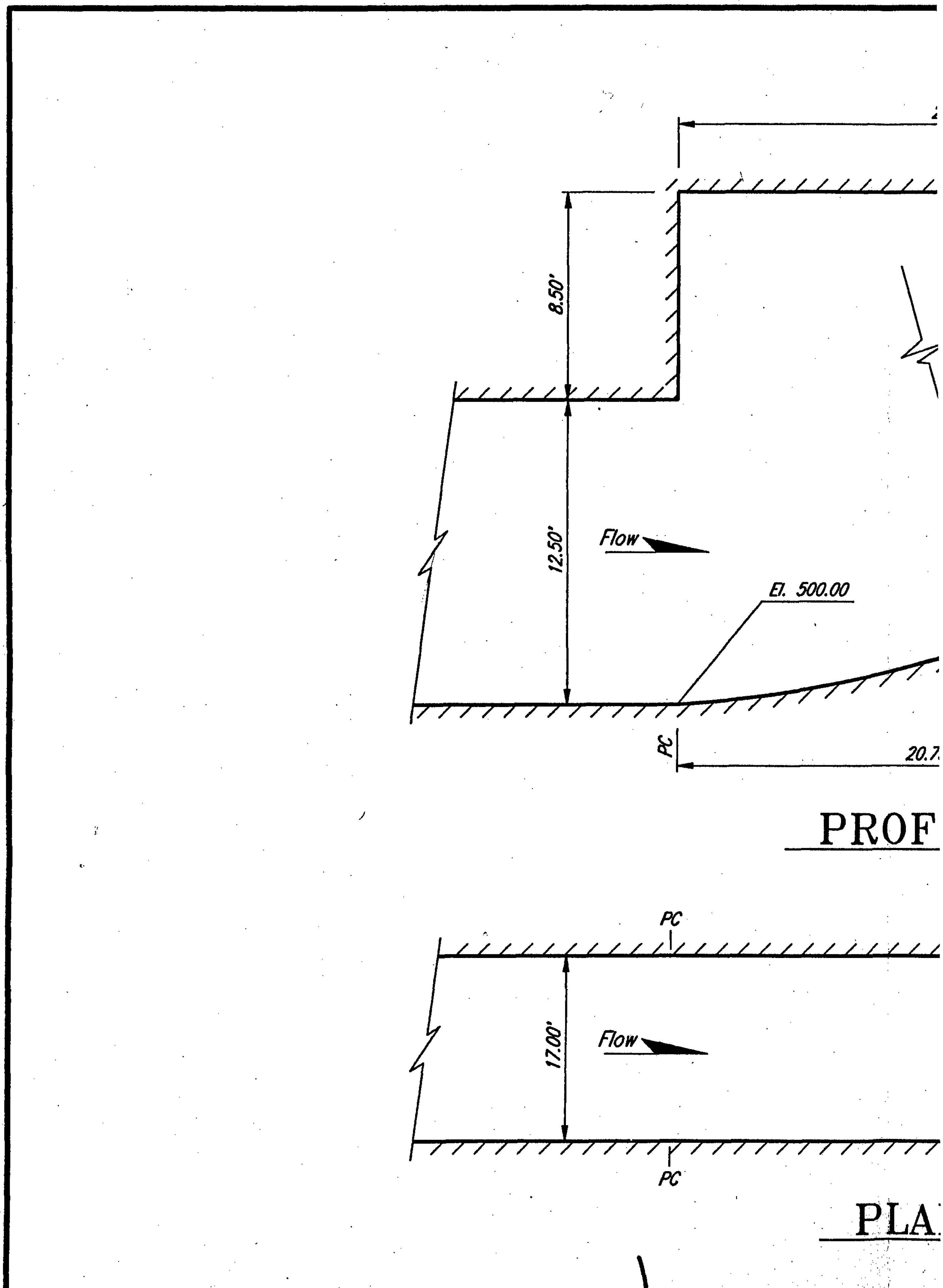
PLAN

AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

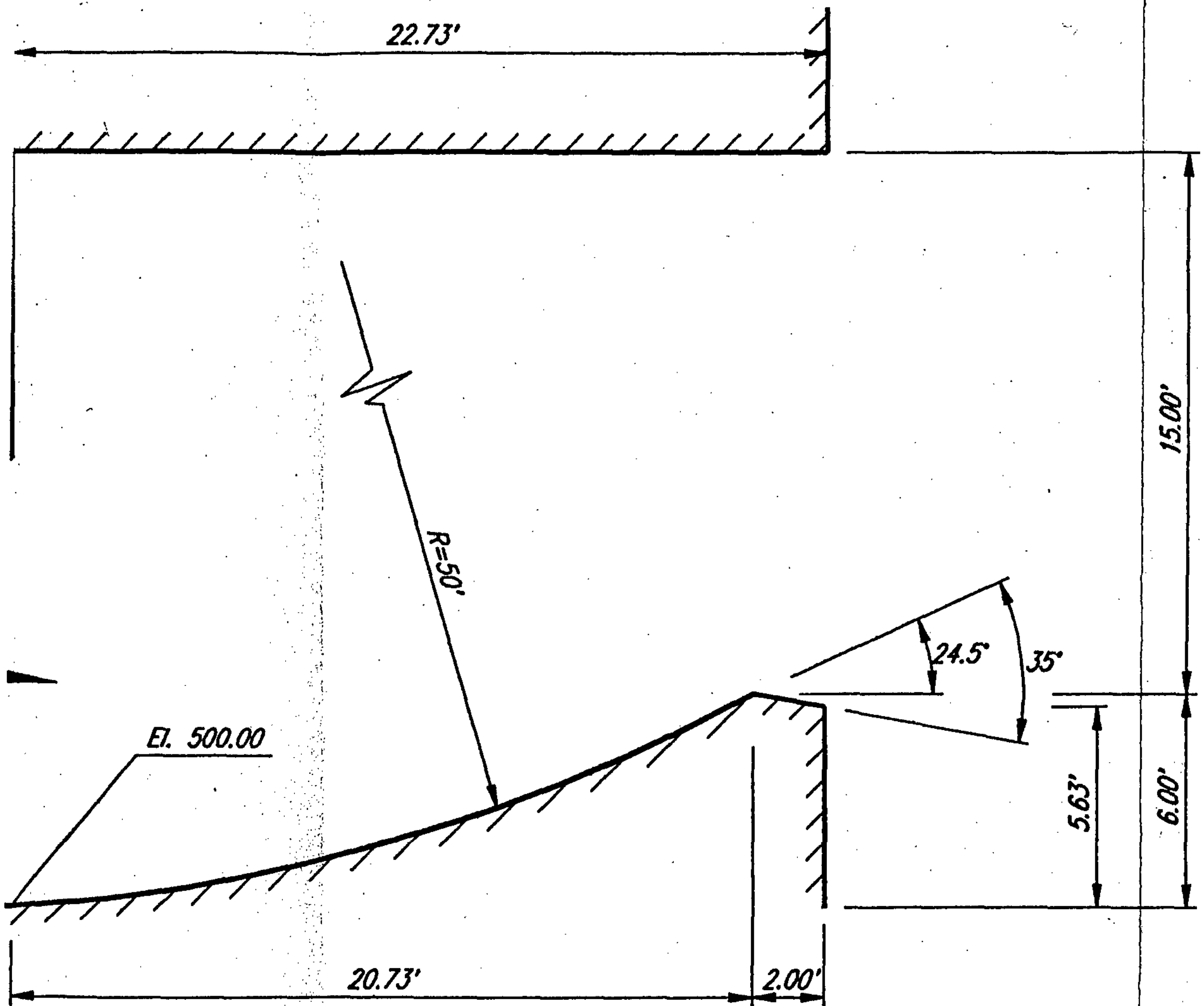
SELECTED PLAN
RM 20.1 DAM
FLOOD CONTROL SLUICE
ENTRANCE AND TRASH RACK GEOMETRY

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
NOVEMBER 1991

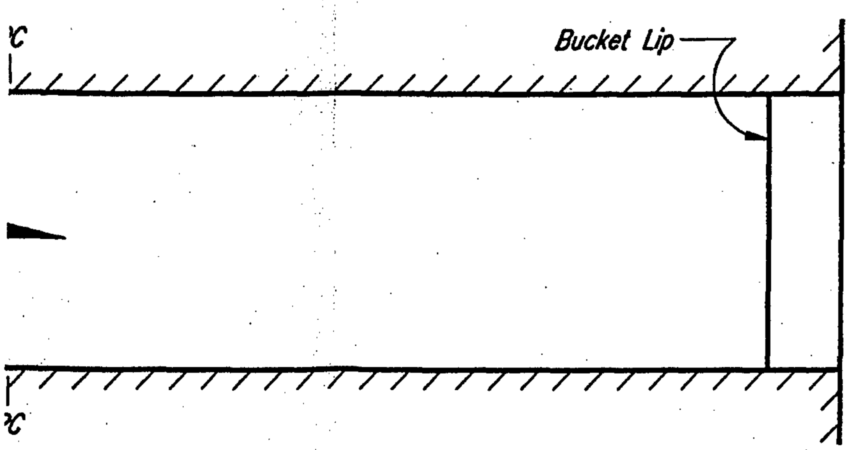
3



8.50'
12.50'
Flow
El. 500.00
PC
PROF
PC
17.00'
Flow
PC
PLA



PROFILE

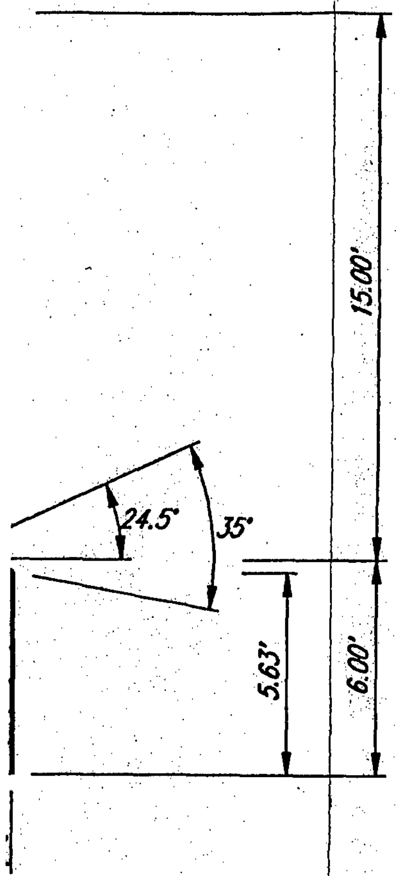


PLAN

AMERICAN RIVER WAT
CALIF

SELECT
RM 20
FLOOD CON
FLIP BUCKET O

SACRAMENTO DISTRICT,
NOVEMB



AMERICAN RIVER WATERSHED INVESTIGATION
CALIFORNIA

SELECTED PLAN
RM 20.1 DAM
FLOOD CONTROL SLUICE
FLIP BUCKET OUTLET GEOMETRY

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
NOVEMBER 1991

3

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

APPENDIX N

CHAPTER 4

M-CACES COST ESTIMATE FOR THE SELECTED PLAN

OCTOBER 1991

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

APPENDIX N

CHAPTER 4

M-CACES COST ESTIMATE
SELECTED PLAN

This chapter contains the cost estimates for the selected plan. Estimates are developed by items required for the separate contracts. Contract cost estimates are then added to develop the total project cost. The contracts are as follows:

Cultural Resource Work
Natomas Levees and Pump Stations
Natomas Recreation
Relocations for the Dam
Foundation and Access Roads
Main Dam

Summaries of costs are given at the beginning of the chapter and then followed by the detailed cost estimates. The estimates are given in the order of total project and then the contracts in the order listed above. Contingencies are added for each separate item rather than applied to the project or contract as a whole. This allows the contingency amount to vary according to the unknowns and sensitivity associated with each separate item. Contingencies vary from 9% to 100%.

COST ESTIMATE
AMERICAN RIVER PROJECT
CALIFORNIA
FEASIBILITY REPORT

COST ESTIMATE

===

To the best of my knowledge the cost estimate was prepared in full compliance with EC 1110-2-263 dated 28 February 1989 and EC 1110-2-538 dated 28 February 1989. Both Engineer Circulars expiration dates have been extended by Civil Works Cost Estimating Guidance Update dated 23 February 1990. The fully funded cost estimate was prepared in full compliance with EC 11-2-157 published in March 1990.

Harvey T. Louis ______________________ Dated: 2 Dec 91
Harvey T Louis
Cost Estimator

______________________ Dated: ______________
Andrew M. Abrate
Chief, Cost Engineering Branch

______________________ Dated: ______________
Lewis A. Whitney
Chief, Engineering Division

______________________ Dated: ______________
Bob Childs
Life Cycle Project Manager

______________________ Dated: ______________
John P. Saia
Chair, Project Review Board

1. TOTAL PROJECT COST SUMMARIES.

The project cost summary fully funded with mid-points of construction identified is shown in Table 1. The cost is based on October 1, 1991 price level. The apportionment of Federal and Non-Federal first costs is based on the criteria contained in the Water Resources Development Act of 1986.

2. BASIS OF FIRST COST.

The detailed estimates of the project first cost with contingencies are shown in Table 2. The estimate is based on the following.

Lands and Damages. These costs are provided by the Sacramento District Real Estate Division based on land studies by the California State Department of Water Resources.

Fish and Wildlife Facilities. These costs are provided by the Sacramento District Environmental Resources Branch. Unit costs per acre for mitigation lands represent cost for site work, plantings, and any required irrigation systems and are based on similar work for habitat restoration done in the district.

Cultural Resources Preservation. This cost is provided by the Sacramento District Environmental Resources Branch and is based on past studies that have indicated that many sites exist in areas that will be impacted. The cost is more than the one percent limit of total Federal appropriation and a waiver will be a part of the Feasibility Report.

PED and Construction Management. These costs are provided by the Sacramento District Design and Construction branches and are based on experience to date on similar projects in the Sacramento District.

Construction. The construction costs were developed using the M-CACES EDITION of COMPOSER Gold. The labor and equipment costs are based on the M-CACES Unit Price Book data base and experience to date on similar projects in the district. The material prices are based on quotes and corresponding bid unit prices received on comparable work in the area. An estimate of how the major construction will be accomplished is as follows.

A. Natomas Levees and Pump Station. (24 months construction)

Levees and Channels. The major cost will be for earthwork. Stripped material will be hauled to a waste area. The material for the levee embankment will be excavated from borrow with an average haul distance of approximately 7 miles over existing roads. Excavation will be by earth excavator and front end loaders and hauling by truck. The material will be placed and compacted using dozers and compaction equipment.

Relocations. The major cost will be for road work. The work can be done using conventional road construction methods and equipment.

Concrete construction is normal concrete work (formwork, placing reinforcement and concrete, and finishing). Concrete is available locally. Delivery can be by truck and placement by pump or direct chute.

Pumping Plant. The major costs will be for machinery and appurtenances. The installation can be done by locally available labor.

B. Dam Foundation and Access Roads. (18 months construction)

Dam Foundation. The major cost will be for earthwork. Excavated material will be hauled approximately one-half mile to a waste area just upstream of the dam. Excavation can be done by dozers, scrapers, excavators and front end loaders and hauling by scrapers or truck. Some blasting may be required. The dumped material can be spread using dozers.

Major concrete work is placement of mass concrete. Concrete cost is based on setting up a concrete plant on site. Delivery can be by truck and placement by pump or direct chute.

Roads. The road work can be done using conventional road construction methods and equipment. Materials required can be obtained from local sources.

C. Main Dam. (36 months construction)

Main Dam. The major cost will be for the main dam concrete. Approximately 4.6 million cubic yards of roller compacted concrete (RCC) and 170,000 cubic yards of conventional concrete will be required. All of the aggregate for the concrete can be obtained from a site approximately 5 miles upstream of the dam. The rock can be mined, crushed to a suitable size, and moved by conveyor to a contractor installed aggregate processing and concrete mixing plant just upstream of the dam. There it can be crushed to the required aggregate size and mixed for use in the RCC or conventional concrete as required. Conveyors can be used to move the mix to the dam. The RCC can be spread and compacted by dozer and compaction equipment in open areas and hand compacted in confined areas. The conventional concrete is normal concrete work (formwork, placing reinforcement and concrete, and finishing). Placement can done by pump or direct chute.

Outlet Works. The major cost is for the placement of approximately 70,000 cubic yards of conventional concrete. The concrete cost is based on setting up a concrete plant on site and placed as described above.

D. Relocations. (24 month construction)

Bridges. The major work will be the construction of four bridges. The Ponderosa, Warner Ravine, and American River North Fork bridges will be of post-tensioned concrete, single box girder and cast-in-place, utilizing a segmental cantilever construction method. Piers will be hollow, cast-in-place tapered box sections on mass concrete shaft foundations. The Highway 49 Viaduct bridge superstructure and piers are similar to the other three bridges except it will be of precast prestressed concrete sections. Materials required are available locally.

Roads. The road work can be done using conventional methods and equipment for road construction.

Utilities. The major cost is for the relocation of electrical lines. The work can be done by local contractors.

E. Recreation. (18 month construction)

Day Use Areas. The major cost will be for roadwork. Construction can be done using conventional methods and equipment. Materials required can be obtained from local sources.

3. CONSTRUCTION CONTINGENCY.

The contingency percentages noted in Table 2 are justified as follows.

Lands

(1.A) Based on minor project design changes, unknown number of condemnation cases, unknown title issues and unknown crediting problems.

(1.B) Based on unknown condemnation settlements, undetected improvements, minor project design changes, unknown splits and market data availability.

Relocations

(2.A) Based on feasibility concepts and quantities.

(2.B) Based on quantities derived from old topography.

(2.C) Based on possibility of change in aggregate source and no project specific concrete mix design studies.

Reservoirs

(3.A) Based on feasibility concepts and quantities.

Dams

(4.A) Based on feasibility concepts and quantities.

(4.B) Based on quantities from old topography and inherent foundation unknowns.

(4.C) Based on no site specific explorations.

(4.D) Based on old topography and no site specific explorations.

(4.E) Based on possibility of change in aggregate source and no project specific concrete mix design studies.

(4.F) Based on need for physical model to define final configuration.

Fish and Wildlife Facilities

(6.A) Based on feasibility concepts and quantities.

Channels and Canals

(9.A) Based on feasibility concepts and quantities.

Levees and Floodwalls

(11.A) Based on feasibility concepts and quantities.

Pumping Plant

(13.A) Based on feasibility concepts and quantities.

Recreation

(14.A) Based on feasibility concepts and quantities.

Permanent Operating Equipment

(20.A) Based on feasibility concepts and quantities.

PED

(30.A) Based on feasibility concepts and quantities.

Construction Management

(31.A) Based on feasibility concepts and quantities.

4. **ANNUAL COST.**
The summary of project annual cost is shown in Table 3.

5. **BASIS OF ANNUAL COST.**
The detailed estimate of project annual cost is shown in Table 4. The cost is based on October 1, 1991 price levels. The interest rate used is 8-3/4 percent with an amortization period of 100 years. Interest during construction was added to the first cost. Annual charges for maintenance and operation are based on experience in the Sacramento District.

SUMMARY COSTS

AMERICAN RIVER WATERSHED INVESTIGATION
SUMMARY OF FIRST COST

TOTAL PROJECT COST

ACCOUNT CODE	ITEM	TOTAL COST
01	LANDS AND DAMAGES	$81,340,000
02	RELOCATIONS	$107,450,000
03	RESERVOIRS	$470,000
04	DAMS	$315,200,000
06	FISH AND WILDLIFE FACILITIES	$9,300,000
08	ROADS, RAILROADS, AND BRIDGES	$2,000,000
09	CHANNELS AND CANALS	$1,020,000
11	LEVEES AND FLOODWALLS	$5,200,000
13	PUMPING PLANT	$4,300,000
14	RECREATION FACILITIES	$1,400,000
18	CULTURAL RESOURCE PRESERVATION	$4,700,000
20	PERMANENT OPERATING EQUIPMENT	$2,900,000
30	ENGINEERING AND DESIGN	$45,340,000
31	CONSTRUCTION MANAGEMENT	$39,840,000
	TOTAL COST FOR PROJECT	$620,460,000
	CREDITABLE EXPENDITURES TO DATE	$77,700,000
	FINAL TOTAL	$698,160,000

AMERICAN RIVER WATERSHED INVESTIGATION
SUMMARY OF FIRST COST

CULTURAL RESOURCE CONTRACT

ACCOUNT CODE	ITEM	TOTAL COST
18	CULTURAL RESOURCE PRESERVATION	$4,700,000
	TOTAL COST FOR CONTRACT	$4,700,000

AMERICAN RIVER WATERSHED INVESTIGATION
SUMMARY OF FIRST COST

NATOMAS LEVEE AND PUMP STATION CONTRACT

ACCOUNT CODE	ITEM	TOTAL COST
01	LANDS AND DAMAGES	$14,020,000
	FED	$320,000
	NFED	$13,700,000
02	RELOCATIONS	$4,010,000
	FED	$0
	NFED	$4,010,000
06	FISH AND WILDLIFE FACILITIES	$5,600,000
09	CHANNELS AND CANALS	$1,020,000
11	LEVEES AND FLOODWALLS	$5,200,000
13	PUMPING PLANT	$4,300,000
30	ENGINEERING AND DESIGN	$3,900,000
	FED	$3,100,000
	NFED	$800,000
31	CONSTRUCTION MANAGEMENT	$1,700,000
	FED	$1,360,000
	NFED	$340,000
	TOTAL NATOMAS LEVEE AND PUMP STATION CONTRACT	$39,750,000

AMERICAN RIVER WATERSHED INVESTIGATION
SUMMARY OF FIRST COST

NATOMAS RECREATION CONTRACT

ACCOUNT CODE	ITEM	TOTAL COST
01	LANDS AND DAMAGES	$6,760,000
	FED	$360,000
	NON-FED	$6,400,000
14	RECREATION FACILITIES	$1,400,000
30	ENGINEERING AND DESIGN	$340,000
31	CONSTRUCTION MANAGEMENT	$270,000
	TOTAL NATOMAS RECREATION CONTRACT	$8,770,000

AMERICAN RIVER WATERSHED INVESTIGATION
SUMMARY OF FIRST COST

DAM RELOCATIONS CONTRACT

ACCOUNT CODE	ITEM	TOTAL COST
02	RELOCATIONS (SUB TOTAL)	$103,440,000
	HIGHWAY 49 BRIDGES	$72,300,000
	PONDEROSA ROAD	$3,200,000
	PONDEROSA BRIDGE	$15,800,000
	UTILITIES	$11,000,000
	LANDS NON-FEDERAL	$990,000
	LANDS FEDERAL	$150,000
30	ENGINEERING AND DESIGN	$10,900,000
31	CONSTRUCTION MANAGEMENT	$8,670,000
	TOTAL DAM RELOCATIONS CONTRACT	$123,010,000

AMERICAN RIVER WATERSHED INVESTIGATION
SUMMARY OF FIRST COST

DAM FOUNDATION CONTRACT

ACCOUNT CODE	ITEM	TOTAL COST
01	LANDS AND DAMAGES	$46,680,000
	FED	$380,000
	NFED	$46,300,000
04	DAMS	$77,600,000
08	ROADS, RAILROADS, AND BRIDGES	$2,000,000
30	ENGINEERING AND DESIGN	$7,600,000
31	CONSTRUCTION MANAGEMENT	$7,100,000
	TOTAL DAM FOUNDATION CONTRACT	$140,980,000

AMERICAN RIVER WATERSHED INVESTIGATION
SUMMARY OF FIRST COST

MAIN DAM CONTRACT

ACCOUNT CODE	ITEM	TOTAL COST
01	LANDS AND DAMAGES	$13,880,000
	FED	$280,000
	NFED	$13,600,000
03	RESERVOIRS	$470,000
04	DAMS	$237,600,000
06	FISH AND WILDLIFE FACILITIES	$3,700,000
20	PERMANENT OPERATING EQUIPMENT	$2,900,000
30	ENGINEERING AND DESIGN	$22,600,000
31	CONSTRUCTION MANAGEMENT	$22,100,000
	TOTAL MAIN DAM CONTRACT	$303,250,000

DETAILED COST ESTIMATES

TOTAL PROJECT COST

TABLE 1

PROJECT: AMERICAN RIVER PROJECT PREPARED BY: SACRAMENTO DISTRICT
LOCATION: CALIFORNIA
DATE PREPARED: 2-Dec-91 Effective Price Date(EPD) 1-Oct-91 REVIEWED & APPROVED BY: ANDY ABRATE BRANCH CHIEF

ACCOUNT NUMBER	ITEM DESCRIPTION	COST (EPD) ($)	CONTING. AMOUNT (EPD) ($) *	%	TOTAL EST EST (EPD) ($) *	MID PT OF CONST	OMB INFL. (+/-)	INFLATED COST AMOUNT ($) *	INFLATED CONTG. AMT. ($) *	FULLY FUNDED COST ($) *
	FEDERAL COSTS									
03---	**RESERVOIRS**	**376,800**	**93,200**	**25%**	**470,000**	**Jun-01**	**48.9%**	**560,000**	**140,000**	**700,000**
04---	**DAMS**	**259,879,370**	**55,320,630**	**21%**	**315,200,000**		**38.6%**	**360,600,000**	**76,400,000**	**437,000,000**
1--	MAIN DAM	226,805,810	47,794,190	21%	274,600,000		38.4%	314,180,000	65,820,000	380,000,000
	Dam Foundation	57,580,960	15,019,040	26%	72,600,000	Sep-98	28.1%	73,750,000	19,250,000	93,000,000
	Main Dam	169,224,850	32,775,150	19%	202,000,000	Jun-01	42.1%	240,430,000	46,570,000	287,000,000
3--	OUTLET WORKS	33,073,560	7,526,440	23%	40,600,000		40.4%	46,420,000	10,580,000	57,000,000
	Dam Foundation	4,153,200	846,800	20%	5,000,000	Sep-98	30.0%	5,400,000	1,100,000	6,500,000
	Main Dam	28,920,360	6,679,640	23%	35,600,000	Jun-01	41.9%	41,020,000	9,480,000	50,500,000
06---	**FISH & WILDLIFE FACILITIES**	**8,503,000**	**797,000**	**9%**	**9,300,000**		**33.3%**	**11,340,000**	**1,060,000**	**12,400,000**
	Natomas Area	5,110,000	490,000	10%	5,600,000	Jun-98	28.6%	6,570,000	630,000	7,200,000
	Main Dam Area	3,393,000	307,000	9%	3,700,000	Jun-01	40.5%	4,770,000	430,000	5,200,000
08---	**ROADS, RAILROADS & BRIDGES**	**1,762,500**	**237,500**	**13%**	**2,000,000**		**30.0%**	**2,290,000**	**310,000**	**2,600,000**
	Main Dam Access Roads	1,762,500	237,500	13%	2,000,000	Sep-98	30.0%	2,290,000	310,000	2,600,000
09---	**CHANNELS AND CANALS**	**887,900**	**132,100**	**15%**	**1,020,000**	**Jun-98**	**27.5%**	**1,130,000**	**170,000**	**1,300,000**
11---	**LEVEES AND FLOODWALLS**	**4,478,150**	**721,850**	**16%**	**5,200,000**	**Jun-98**	**28.8%**	**5,770,000**	**930,000**	**6,700,000**
13---	**PUMPING PLANT**	**3,736,978**	**563,022**	**15%**	**4,300,000**	**Jun-98**	**27.9%**	**4,780,000**	**720,000**	**5,500,000**
14---	**RECREATION FACILITIES**	**1,245,575**	**154,425**	**12%**	**1,400,000**	**Sep-98**	**28.6%**	**1,600,000**	**200,000**	**1,800,000**
20---	**PERMANENT OPERATING EQUIP.**	**2,320,000**	**580,000**	**25%**	**2,900,000**	**Jun-01**	**41.4%**	**3,280,000**	**820,000**	**4,100,000**
	Subtotal Construction	283,190,273	58,599,727	21%	341,790,000			391,350,000	80,750,000	472,100,000
01---	**LANDS AND DAMAGES**	**1,158,280**	**181,720**	**16%**	**1,340,000**		**41.8%**	**1,640,000**	**260,000**	**1,900,000**
	Natomas Levees/Pump Sta	275,410	44,590	16%	320,000		31.3%	360,000	60,000	420,000
	Dam Foundation	333,050	46,950	14%	380,000		31.6%	440,000	60,000	500,000
	Main Dam	240,470	39,530	16%	280,000		57.1%	380,000	60,000	440,000
	Natomas Recreation	309,350	50,650	16%	360,000		50.0%	460,000	80,000	540,000
02---	**RELOCATIONS-Real Estate**	**134,640**	**15,360**	**11%**	**150,000**		**33.3%**	**180,000**	**20,000**	**200,000**
18---	**CULTURAL RESOURCE PRES.**	**4,230,000**	**470,000**	**11%**	**4,700,000**	**Oct-96**	**21.3%**	**5,130,000**	**570,000**	**5,700,000**
30---	**PLANNING, ENGR. & DESIGN**	**27,423,198**	**6,216,802**	**23%**	**33,640,000**		**28.1%**	**35,110,000**	**7,990,000**	**43,100,000**
	Natomas Levees/Pump Sta	2,837,568	262,432	9%	3,100,000		25.8%	3,570,000	330,000	3,900,000
	Dam Foundation	6,133,900	1,466,100	24%	7,600,000		21.2%	7,430,000	1,780,000	9,210,000
	Main Dam	18,144,700	4,455,300	25%	22,600,000		30.8%	23,730,000	5,840,000	29,570,000
	Natomas Recreation	307,030	32,970	11%	340,000		23.5%	380,000	40,000	420,000
31---	**CONSTRUCTION MANAGEMENT**	**27,746,990**	**3,083,010**	**11%**	**30,830,000**		**68.0%**	**46,630,000**	**5,170,000**	**51,800,000**
	Natomas Levees/Pump Sta	1,200,777	159,223	13%	1,360,000		49.3%	1,790,000	240,000	2,030,000
	Dam Foundation	6,380,760	719,240	11%	7,100,000		54.9%	9,880,000	1,120,000	11,000,000
	Main Dam	19,922,651	2,177,349	11%	22,100,000		73.4%	34,560,000	3,770,000	38,330,000
	Natomas Recreation	242,802	27,198	11%	270,000		63.0%	400,000	40,000	440,000

TABLE 1

ROJECT: AMERICAN RIVER PROJECT PREPARED BY: SACRAMENTO DISTRICT
OCATION: CALIFORNIA
ATE PREPARED: 2-Dec-91 Effective Price Date(EPD) 1-Oct-91 REVIEWED & APPROVED BY: ANDY ABRATE BRANCH CHIEF

CCOUNT UMBER	ITEM DESCRIPTION	COST (EPD) ($)	CONTING. AMOUNT (EPD) ($) *	%	TOTAL EST EST (EPD) ($) *	MID PT OF CONST	OMB INFL. (+/-)	INFLATED COST AMOUNT ($) *	INFLATED CONTG. AMT. ($) *	FULLY FUNDED COST ($) *
	FEDERAL COSTS									
EDERAL/NON-FEDERAL CONTRIBUTION										
	SUBTOTAL	343,883,381	68,566,619	20%	412,450,000			480,040,000	94,760,000	574,800,000
ON-FEDERAL CONTRIBUTION		27,850,000	4,600,000	17%	32,450,000			36,340,000	6,160,000	42,500,000
OTAL FEDERAL COSTS		**316,033,381**	**63,966,619**	**20%**	**380,000,000**			**443,700,000**	**88,600,000**	**532,300,000**
	NON-FEDERAL COSTS									
2---	**RELOCATIONS-Construction**	**93,469,339**	**12,840,661**	**14%**	**106,310,000**		**28.7%**	**120,300,000**	**16,500,000**	**136,800,000**
1--	ROADS	83,550,629	11,649,371	14%	95,200,000		28.4%	107,260,000	14,940,000	122,200,000
	Natomas Levees	3,381,790	518,210	15%	3,900,000		17.9%	3,990,000	610,000	4,600,000
	Warner Ravine Bridge	23,737,238	3,062,762	13%	26,800,000		29.9%	30,820,000	3,980,000	34,800,000
	American R.N.Fork Bridge	31,617,076	4,082,924	13%	35,700,000		28.6%	40,660,000	5,240,000	45,900,000
	Viaduct Bridge	8,657,620	1,142,380	13%	9,800,000		26.5%	10,950,000	1,450,000	12,400,000
	Ponderosa Road	2,778,185	421,815	15%	3,200,000		31.3%	3,650,000	550,000	4,200,000
	Ponderosa Bridge	13,378,720	2,421,280	18%	15,800,000		28.5%	17,190,000	3,110,000	20,300,000
2--	RAILROADS	32,750	7,250	22%	40,000	Apr-96	25.0%	40,000	10,000	50,000
	Natomas Levees									
3--	CEMETERY, UTILITY, STRUCT.	9,885,960	1,184,040	12%	11,070,000		31.4%	13,000,000	1,550,000	14,550,000
	Natomas Levees	57,840	12,160	21%	70,000	Apr-96	14.3%	70,000	10,000	80,000
	Highway 49 Utilities	9,828,120	1,171,880	12%	11,000,000		31.5%	12,930,000	1,540,000	14,470,000
	Subtotal Construction	93,469,339	12,840,661	14%	106,310,000			120,300,000	16,500,000	136,800,000
1---	**LANDS AND DAMAGES**	**66,034,200**	**13,965,800**	**21%**	**80,000,000**		**20.0%**	**79,240,000**	**16,760,000**	**96,000,000**
	Natomas Levees/Pump Sta	11,541,300	2,158,700	19%	13,700,000		22.6%	14,150,000	2,650,000	16,800,000
	Dam Foundation	37,823,300	8,476,700	22%	46,300,000		16.8%	44,170,000	9,930,000	54,100,000
	Main Dam	11,176,850	2,423,150	22%	13,600,000		25.7%	14,050,000	3,050,000	17,100,000
	Natomas Recreation	5,492,750	907,250	17%	6,400,000		25.0%	6,870,000	1,130,000	8,000,000
2---	**RELOCATIONS-Real Estate**	**821,480**	**168,520**	**21%**	**990,000**		**21.2%**	**990,000**	**210,000**	**1,200,000**
A--	ROADS	726,040	153,960	21%	880,000		20.5%	870,000	190,000	1,060,000
C--	CEMETERY/UTILITY/STRUCT	95,440	14,560	15%	110,000		27.3%	120,000	20,000	140,000
3---	**PLANNING, ENGR. & DESIGN**	**9,521,372**	**2,178,628**	**23%**	**11,700,000**		**20.5%**	**11,480,000**	**2,620,000**	**14,100,000**
	Natomas Levees	705,872	94,128	13%	800,000		18.8%	840,000	110,000	950,000
	Hwy 49/Ponderosa/Utility	8,815,500	2,084,500	24%	10,900,000		20.6%	10,640,000	2,510,000	13,150,000
3---	**CONSTRUCTION MANAGEMENT**	**8,102,412**	**907,588**	**11%**	**9,010,000**		**26.5%**	**10,250,000**	**1,150,000**	**11,400,000**
	Natomas Levees	298,704	41,296	14%	340,000		17.6%	350,000	50,000	400,000
	Hwy 49/Ponderosa/Utility	7,803,708	866,292	11%	8,670,000		26.9%	9,900,000	1,100,000	11,000,000
SBTOTAL NON-FEDERAL		177,948,803	30,061,197	17%	208,010,000			222,260,000	37,240,000	259,500,000
NN-FEDERAL CONTRIBUTION		27,850,000	4,600,000	17%	32,450,000			36,340,000	6,160,000	42,500,000
TAL NON-FEDERAL COSTS		**205,798,803**	**34,661,197**	**17%**	**240,460,000**			**258,600,000**	**43,400,000**	**302,000,000**
TAL COST FEDERAL/NON-FEDERAL		**521,832,184**	**98,627,816**	**19%**	**620,460,000**			**702,300,000**	**132,000,000**	**834,300,000**
TTAL CREDITABLE EXPENDITURES TO DATE					77,700,000					77,700,000
TTAL PROJECT COST INCLUDING CREDITABLE EXPENDITURE					698,160,000					912,000,000

TABLE 1

TOTAL - CONTRACT F **** TOTAL PROJECT COST SUMMARIES **** PAGE 1 OF 1

PROJECT: AMERICAN RIVER PROJECT PREPARED BY: SACRAMENTO DISTRICT
LOCATION: CALIFORNIA
DATE PREPARED: 2-Dec-91 Effective Price Date(EPD) 1-Oct-91 REVIEWED & APPROVED BY: ANDY ABRATE BRANCH CHIEF

ACCOUNT NUMBER	ITEM DESCRIPTION	COST (EPD) ($)	CONTING. AMOUNT (EPD) ($) *	%	TOTAL EST EST (EPD) ($) *	MID PT OF CONST	OMB INFL. (+/-)	INFLATED COST AMOUNT ($) *	INFLATED CONTG. AMT. ($) *	FULLY FUNDED COST ($) *
	Cultural Resources Preservation									
	FEDERAL COSTS									
18---	CULTURAL RESOURCE PRES.	4,230,000	470,000	11%	4,700,000	Oct-96	21.3%	5,130,000	570,000	5,700,000

TABLE 1

TOTAL - CONTRACT A	**** TOTAL PROJECT COST SUMMARIES ****	PAGE 1 OF 1

PROJECT: AMERICAN RIVER PROJECT — PREPARED BY: SACRAMENTO DISTRICT
LOCATION: CALIFORNIA
DATE PREPARED: 2-Dec-91 Effective Price Date(EPD) 1-Oct-91 REVIEWED & APPROVED BY: ANDY ABRATE BRANCH CHIEF

ACCOUNT NUMBER	ITEM DESCRIPTION	COST (EPD) ($)	CONTING. AMOUNT (EPD) ($) *	%	TOTAL EST EST (EPD) ($) *	MID PT OF CONST	OMB INFL. (+/-)	INFLATED COST AMOUNT ($) *	INFLATED CONTG. AMT. ($) *	FULLY FUNDED COST ($) *
	Natomas Levees and Pump Station									
	FEDERAL COSTS									
06---	**FISH & WILDLIFE FACILITIES**	**5,110,000**	**490,000**	**10%**	**5,600,000**		**28.6%**	**6,570,000**	**630,000**	**7,200,000**
3--	WILDLIFE FACILITIES & SANCTUARIES	5,110,000	490,000	10%	5,600,000	Jun-98	28.6%	6,570,000	630,000	7,200,000
09---	**CHANNELS AND CANALS**	**887,900**	**132,100**	**15%**	**1,020,000**	**Jun-98**	**27.5%**	**1,130,000**	**170,000**	**1,300,000**
11---	**LEVEES AND FLOODWALLS**	**4,478,150**	**721,850**	**16%**	**5,200,000**	**Jun-98**	**28.8%**	**5,770,000**	**930,000**	**6,700,000**
13---	**PUMPING PLANT**	**3,736,978**	**563,022**	**15%**	**4,300,000**	**Jun-98**	**27.9%**	**4,780,000**	**720,000**	**5,500,000**
	Subtotal Construction	14,213,028	1,906,972	13%	16,120,000			18,250,000	2,450,000	20,700,000
01---	**LANDS AND DAMAGES**	**275,410**	**44,590**	**16%**	**320,000**		**31.3%**	**360,000**	**60,000**	**420,000**
30---	**PLANNING, ENGR. & DESIGN**	**2,837,568**	**262,432**	**9%**	**3,100,000**		**25.8%**	**3,570,000**	**330,000**	**3,900,000**
31---	**CONSTRUCTION MANAGEMENT**	**1,200,777**	**159,223**	**13%**	**1,360,000**		**49.3%**	**1,790,000**	**240,000**	**2,030,000**
	SUBTOTAL FEDERAL & NON-FEDERAL CONTRIBUTION	18,526,783	2,373,217	13%	20,900,000			23,970,000	3,080,000	27,050,000
	NON-FEDERAL COSTS									
02---	**RELOCATIONS**	**3,472,380**	**537,620**	**15%**	**4,010,000**		**18.0%**	**4,100,000**	**630,000**	**4,730,000**
	Construction Activities									
1--	ROADS	3,381,790	518,210	15%	3,900,000	Apr-96	17.9%	3,990,000	610,000	4,600,000
2--	RAILROADS	32,750	7,250	22%	40,000	Apr-96	25.0%	40,000	10,000	50,000
3--	CEMETERY/UTILITY/STRUCT	57,840	12,160	21%	70,000	Apr-96	14.3%	70,000	10,000	80,000
	Subtotal Construction	3,472,380	537,620	15%	4,010,000			4,100,000	630,000	4,730,000
01---	**LANDS AND DAMAGES**	**11,541,300**	**2,158,700**	**19%**	**13,700,000**		**22.6%**	**14,150,000**	**2,650,000**	**16,800,000**
30---	**PLANNING, ENGR. & DESIGN**	**705,872**	**94,128**	**13%**	**800,000**		**18.8%**	**840,000**	**110,000**	**950,000**
31---	**CONSTRUCTION MANAGEMENT**	**298,704**	**41,296**	**14%**	**340,000**		**17.6%**	**350,000**	**50,000**	**400,000**
	SUBTOTAL NON-FEDERAL	16,018,256	2,831,744	18%	18,850,000			19,440,000	3,440,000	22,880,000
	TOTAL FEDERAL AND NON-FEDERAL COSTS	**34,545,039**	**5,204,961**	**15%**	**39,750,000**			**43,410,000**	**6,520,000**	**49,930,000**

TABLE 1

PROJECT: AMERICAN RIVER PROJECT PREPARED BY: SACRAMENTO DISTRICT
LOCATION: CALIFORNIA
DATE PREPARED: 2-Dec-91 Effective Price Date(EPD) 1-Oct-91 REVIEWED & APPROVED BY: ANDY ABRATE BRANCH CHIEF

ACCOUNT NUMBER	ITEM DESCRIPTION	COST (EPD) ($)	CONTING. AMOUNT (EPD) ($) *	%	TOTAL EST EST (EPD) ($) *	MID PT OF CONST	OMB INFL. (+/-)	INFLATED COST AMOUNT ($) *	INFLATED CONTG. AMT. ($) *	FULLY FUNDED COST ($) *
	Natomas Recreation									
	FEDERAL COSTS									
14---	**RECREATION FACILITIES**	1,245,575	154,425	12%	1,400,000	Sep-98	28.6%	1,600,000	200,000	1,800,000
	Subtotal Construction	1,245,575	154,425	12%	1,400,000			1,600,000	200,000	1,800,000
01---	**LANDS AND DAMAGES**	309,350	50,650		360,000		50.0%	460,000	80,000	540,000
30---	**PLANNING, ENGR. & DESIGN**	307,030	32,970	11%	340,000		23.5%	380,000	40,000	420,000
31---	**CONSTRUCTION MANAGEMENT**	242,802	27,198	11%	270,000		63.0%	400,000	40,000	440,000
SUBTOTAL FEDERAL & NON-FEDERAL CONTRIBUTION		2,104,757	265,243	13%	2,370,000			2,840,000	360,000	3,200,000
	NON-FEDERAL COSTS									
01---	**LANDS AND DAMAGES**	5,492,750	907,250	17%	6,400,000		25.0%	6,870,000	1,130,000	8,000,000
SUBTOTAL NON-FEDERAL		5,492,750	907,250	17%	6,400,000			6,870,000	1,130,000	8,000,000
TOTAL FEDERAL AND NON-FEDERAL COSTS		7,597,507	1,172,493	15%	8,770,000			9,710,000	1,490,000	11,200,000

TABLE 1

TOTAL - CONTRACT B **** TOTAL PROJECT COST SUMMARIES ****

PROJECT: AMERICAN RIVER PROJECT PREPARED BY: SACRAMENTO DISTRICT
LOCATION: CALIFORNIA
DATE PREPARED: 2-Dec-91 Effective Price Date(EPD) 1-Oct-91 REVIEWED & APPROVED BY: ANDY ABRATE BRANCH CHIEF

ACCOUNT NUMBER	ITEM DESCRIPTION	COST (EPD) ($)	CONTING. AMOUNT (EPD) ($) *	%	TOTAL EST EST (EPD) ($) *	MID PT OF CONST	OMB INFL. (+/-)	INFLATED COST AMOUNT ($) *	INFLATED CONTG. AMT. ($) *	FULLY FUNDED COST ($) *
	Dam Foundation and Access Roads									
	FEDERAL COSTS									
04---	**DAMS**	**61,734,160**	**15,865,840**	**26%**	**77,600,000**		**28.2%**	**79,150,000**	**20,350,000**	**99,500,000**
1--	MAIN DAM	57,580,960	15,019,040	26%	72,600,000	Sep-98	28.1%	73,750,000	19,250,000	93,000,000
3--	OUTLET WORKS	4,153,200	846,800	20%	5,000,000	Sep-98	30.0%	5,400,000	1,100,000	6,500,000
08---	**ROADS, RAILROADS & BRIDGES**	**1,762,500**	**237,500**	**13%**	**2,000,000**		**30.0%**	**2,290,000**	**310,000**	**2,600,000**
2--	ROADS (Incl. Bridges)	1,762,500	237,500	13%	2,000,000	Sep-98	30.0%	2,290,000	310,000	2,600,000
	Subtotal Construction	63,496,660	16,103,340	25%	79,600,000			81,440,000	20,660,000	102,100,000
01---	**LANDS AND DAMAGES**	**333,050**	**46,950**	**14%**	**380,000**		**31.6%**	**440,000**	**60,000**	**500,000**
30---	**PLANNING, ENGR. & DESIGN**	**6,133,900**	**1,466,100**	**24%**	**7,600,000**		**21.2%**	**7,430,000**	**1,780,000**	**9,210,000**
31---	**CONSTRUCTION MANAGEMENT**	**6,380,760**	**719,240**	**11%**	**7,100,000**		**54.9%**	**9,880,000**	**1,120,000**	**11,000,000**
	SUBTOTAL FEDERAL & NON-FEDERAL CONTRIBUTION	76,344,370	18,335,630	24%	94,680,000			99,190,000	23,620,000	122,810,000
	NON-FEDERAL COSTS									
01---	**LANDS AND DAMAGES**	**37,823,300**	**8,476,700**	**22%**	**46,300,000**		**16.8%**	**44,170,000**	**9,930,000**	**54,100,000**
	SUBTOTAL NON-FEDERAL	37,823,300	8,476,700	22%	46,300,000			44,170,000	9,930,000	54,100,000
	TOTAL FEDERAL AND NON-FEDERAL COSTS	**114,167,670**	**26,812,330**	**23%**	**140,980,000**			**143,360,000**	**33,550,000**	**176,910,000**

TABLE 1

TOTAL - CONTRACT C **** TOTAL PROJECT COST SUMMARIES ****

PROJECT: AMERICAN RIVER PROJECT PREPARED BY: SACRAMENTO DISTRICT
LOCATION: CALIFORNIA
DATE PREPARED: 2-Dec-91 Effective Price Date(EPD) 1-Oct-91 REVIEWED & APPROVED BY: ANDY ABRATE BRANCH CHIEF

Main Dam

ACCOUNT NUMBER	ITEM DESCRIPTION	COST (EPD) ($)	CONTING. AMOUNT (EPD) ($) *	%	TOTAL EST EST (EPD) ($) *	MID PT OF CONST	OMB INFL. (+/-)	INFLATED COST AMOUNT ($) *	INFLATED CONTG. AMT. ($) *	FULLY FUNDED COST ($) *
	FEDERAL COSTS									
03---	RESERVOIRS	376,800	93,200	25%	470,000	Jun-01	48.9%	560,000	140,000	700,000
04---	DAMS	198,145,210	39,454,790	20%	237,600,000			281,450,000	56,050,000	337,500,000
1--	MAIN DAM	169,224,850	32,775,150	19%	202,000,000	Jun-01	42.1%	240,430,000	46,570,000	287,000,000
3--	OUTLET WORKS	28,920,360	6,679,640	23%	35,600,000	Jun-01	41.9%	41,020,000	9,480,000	50,500,000
06---	FISH & WILDLIFE FACILITIES	3,393,000	307,000	9%	3,700,000		40.5%	4,770,000	430,000	5,200,000
3--	WILDLIFE FACILITIES & SANCTUARIES	3,393,000	307,000	9%	3,700,000	Jun-01	40.5%	4,770,000	430,000	5,200,000
20---	PERMANENT OPERATING EQUIP.	2,320,000	580,000	25%	2,900,000	Jun-01	41.4%	3,280,000	820,000	4,100,000
	Subtotal Construction	204,235,010	40,434,990	20%	244,670,000			290,060,000	57,440,000	347,500,000
01---	LANDS AND DAMAGES	240,470	39,530	16%	280,000		57.1%	380,000	60,000	440,000
30---	PLANNING, ENGR. & DESIGN	18,144,700	4,455,300	25%	22,600,000		30.8%	23,730,000	5,840,000	29,570,000
31---	CONSTRUCTION MANAGEMENT	19,922,651	2,177,349	11%	22,100,000		73.4%	34,560,000	3,770,000	38,330,000
SUBTOTAL FEDERAL & NON-FEDERAL CONTRIBUTION		242,542,831	47,107,169	19%	289,650,000			348,730,000	67,110,000	415,840,000
	NON-FEDERAL COSTS									
01---	LANDS AND DAMAGES	11,176,850	2,423,150	22%	13,600,000		25.7%	14,050,000	3,050,000	17,100,000
SUBTOTAL NON-FEDERAL		11,176,850	2,423,150	22%	13,600,000			14,050,000	3,050,000	17,100,000
TOTAL FEDERAL AND NON-FEDERAL COSTS		253,719,681	49,530,319	20%	303,250,000			362,780,000	70,160,000	432,940,000

TABLE 1

TOTAL - CONTRACT D	**** TOTAL PROJECT COST SUMMARIES ****	

PROJECT: AMERICAN RIVER PROJECT — PREPARED BY: SACRAMENTO DISTRICT
LOCATION: CALIFORNIA
DATE PREPARED: 2-Dec-91 — Effective Price Date(EPD) 1-Oct-91 — REVIEWED & APPROVED BY: ANDY ABRATE BRANCH CHIEF

ACCOUNT NUMBER	ITEM DESCRIPTION	COST (EPD) ($)	CONTING. AMOUNT (EPD) ($) *	%	TOTAL EST EST (EPD) ($) *	MID PT OF CONST	OMB INFL. (+/-)	INFLATED COST AMOUNT ($) *	INFLATED CONTG. AMT. ($) *	FULLY FUNDED COST ($) *
	Highway 49 / Ponderosa / Utilities Relocations									
	FEDERAL COSTS									
02---	**RELOCATIONS**	**134,640**	**15,360**	**11%**	**150,000**			**180,000**	**20,000**	**200,000**
	Real Estate Activities									
A--	ROADS	134,640	15,360	11%	150,000		33.3%	180,000	20,000	200,000
SUBTOTAL FEDERAL & NON-FEDERAL CONTRIBUTION		134,640	15,360	11%	150,000			180,000	20,000	200,000
	NON-FEDERAL COSTS									
02---	**RELOCATIONS**	**89,996,959**	**12,303,041**	**14%**	**102,300,000**		**29.1%**	**116,200,000**	**15,870,000**	**132,070,000**
	Construction Activities									
1--	ROADS	80,168,839	11,131,161	14%	91,300,000		28.8%	103,270,000	14,330,000	117,600,000
	Warner Ravine Bridge	23,737,238	3,062,762	13%	26,800,000		29.9%	30,820,000	3,980,000	34,800,000
	American Riv.N.Fork Bridge	31,617,076	4,082,924	13%	35,700,000		28.6%	40,660,000	5,240,000	45,900,000
	Viaduct Bridge	8,657,620	1,142,380	13%	9,800,000		26.5%	10,950,000	1,450,000	12,400,000
	Ponderosa Road	2,778,185	421,815	15%	3,200,000		31.3%	3,650,000	550,000	4,200,000
	Ponderosa Bridge	13,378,720	2,421,280	18%	15,800,000		28.5%	17,190,000	3,110,000	20,300,000
3--	CEMETERY/UTILITY/STRUCT	9,828,120	1,171,880	12%	11,000,000		31.5%	12,930,000	1,540,000	14,470,000
	Subtotal Construction	89,996,959	12,303,041	14%	102,300,000			116,200,000	15,870,000	132,070,000
02---	**RELOCATIONS**	**821,480**	**168,520**	**21%**	**990,000**		**21.2%**	**990,000**	**210,000**	**1,200,000**
	Real Estate Activities									
A--	ROADS	726,040	153,960	21%	880,000		20.5%	870,000	190,000	1,060,000
C--	CEMETERY/UTILITY/STRUCT	95,440	14,560	15%	110,000		27.3%	120,000	20,000	140,000
30---	**PLANNING, ENGR. & DESIGN**	**8,815,500**	**2,084,500**	**24%**	**10,900,000**		**20.6%**	**10,640,000**	**2,510,000**	**13,150,000**
31---	**CONSTRUCTION MANAGEMENT**	**7,803,708**	**866,292**	**11%**	**8,670,000**		**26.9%**	**9,900,000**	**1,100,000**	**11,000,000**
SUBTOTAL NON-FEDERAL		107,437,647	15,422,353	14%	122,860,000			137,730,000	19,690,000	157,420,000
TOTAL FEDERAL AND NON-FEDERAL COSTS		**107,572,287**	**15,437,713**	**14%**	**123,010,000**			**137,910,000**	**19,710,000**	**157,620,000**

CULTURAL RESOURCE WORK CONTRACT

COST ESTIMATE

DETAILED ESTIMATE OF FIRST COST   TABLE 2   F1

ACCOUNT NUMBER	ITEM	QUANTITY	UNIT	UNIT PRICE $	AMOUNT $	CONTINGENCY $ *	CONTINGENCY % *	REASON
	Effective Price Date(EPD) 1-Oct-91 Cultural Resources					* (Figures Rounded)		
	FEDERAL							
18.-.-.-	**CULTURAL RESOURCE PRESERVATION**							
18.0.1.-	Identification, Data Analysis and Reports:	1	JOB	329,000	329,000	65,800	20.0	
18.0.2.-	Recover and Remove Artifacts:	1	JOB	141,000	141,000	28,200	20.0	
18.0.3.-	Preservation on Site:							
18.0.3.B	Site Work							
	Special Excavation	1	JOB	3,760,000	3,760,000	376,000	10.0	
	Subtotal, Construction Costs:			$	4,230,000			
	Contingency rounded to an average of	11.1 % +/- *			$	470,000		
18.-.-.-	**CULTURAL RESOURCE PRESERVATION**		**TOTAL:**		**$**	**4,700,000**		

NATOMAS LEVEES AND PUMP STATIONS CONTRACT

COST ESTIMATE

DETAILED ESTIMATE OF FIRST COST TABLE 2 A1

ACCOUNT NUMBER	ITEM	QUANTITY	UNIT	UNIT PRICE $	AMOUNT $	CONTINGENCY $ *	CONTINGENCY % *	REASON
	Effective Price Date(EPD) 1-Oct-91					* (Figures Rounded)		
	Natomas Levees and Pump Station							
	FEDERAL							
01.-.-.-	**LANDS AND DAMAGES**							
01.B.-.-	POST-AUTHORIZATION PLANNING							
01.B.1.-	Develop Cost Estimate	87	MD	440.00	38,280			
01.B.3.-	Real estate design Memorandum	20	MD	450.00	9,000			
01.B.4.-	Evaluate Sponsor Capability	3	MD	530.00	1,590			
01.B.8.-	All Other	163	MD	440.00	71,720			
01.B.9.-	Contingencies					18,000		
01.C.-.-	LOCAL COOPERATION AGREEMENT							
01.C.2.-	Final LCA	4	MD	500.00	2,000			
01.D.-.-	ACQUISITIONS							
01.D.1.-	Attorney's Opinion							
01.D.1.F	Review for Compliance	6	MD	530.00	3,180			
01.D.2.-	Mapping, Survey and Tract Ownership							
01.D.2.D	Prepare Documents	5	MD	360.00	1,800			
01.D.2.F	Review for Compliance	7	MD	360.00	2,520			
01.D.2.Z	All Other	22	MD	350.00	7,700			
01.D.4.-	Negotiations and Closing							
01.D.4.F	Review for Compliance	40	MD	400.00	16,000			
01.D.4.Z	All Other	72	MD	460.00	33,120			
01.D.9.-	Contingencies					9,700		
01.F.-.-	APPRAISALS							
01.F.1.-	Staff Appraisals							
01.F.1.Z	All Other	2	MD	500.00	1,000			
01.F.2.-	Contract Appraisals							
01.F.2.Z	All Other	165	MD	500.00	82,500			
01.F.9.-	Contingencies					12,500		
01.H.-.-	RELOCATION ASSISTANCE							
01.H.1.-	PL 91-646							
01.H.1.D	Prepare Documents	5	MD	500.00	2,500			
01.H.1.F	Review for Compliance	5	MD	500.00	2,500			
01.H.9.-	Contingencies					800		
	Subtotal, Construction Costs:			$	275,410			
	Contingency rounded to an average of	16.2 % +/- *			$	44,590		(1.A)
01.-.-.-	**LANDS AND DAMAGES**		**TOTAL:**		$	320,000		
06.-.-.-	**FISH AND WILDLIFE FACILITIES**							
06.3.-.-	**WILDLIFE FACILITIES AND SANCTUARIES**							
06.3.B.-	Habitat and Feeding Facilities:							
06.3.3.B	Site Work							
	Wetland Creation	151	Acre	24,000	3,624,000	362,400		
	Forage Creation	101	Acre	10,000	1,010,000	101,000		
	Uplands Creation	28	Acre	17,000	476,000	47,600		
	Subtotal, Construction Costs:			$	5,110,000			
	Contingency rounded to an average of	9.6 % +/- *			$	490,000		(6.A)
06.3.-.-	**WILDLIFE FACILITIES AND SANCTUARIES**		**TOTAL:**		$	5,600,000		

DETAILED ESTIMATE OF FIRST COST TABLE 2 A2
Natomas Levees and Pump Station

09.-.-.-	**CHANNELS AND CANALS**						
09.0.2.-	Channels:						
09.0.2.B	Site Work						
	Clearing and Grubbing	30 AC	1,100.00	33,000	4,950	15.0	
	Excavation, Common	158,000 CY	4.00	632,000	100,000	15.8	
	Slope Treatment						
	Gravel Bedding	220 TON	16.00	3,520	500	14.2	
	Riprap	660 TON	23.00	15,180	2,300	15.2	
	Grass Seeding	30 AC	1,200.00	36,000	5,400	15.0	
09.0.3.-	Canals:						
09.0.3.C	Concrete						
	Concrete in Place Including Cement (10-10' x 8' H CBC)	440 CY	330.00	145,200	20,000	13.8	
	Reinforcing Steel	46,000 Lbs	0.50	23,000	3,000	13.0	
	Subtotal, Construction Costs:		$	887,900			
	Contingency rounded to an average of	14.9 % +/- *		$	132,100		(9.A)
09.-.-.-	**CHANNELS AND CANALS**		**TOTAL:**	$	**1,020,000**		
11.-.-.-	**LEVEES AND FLOODWALLS**						
	NATOMAS AREA LEVEES						
11.0.A.-	Mob., Demob. and Preparatory Work:	1 LS	80,000.00	80,000	16,000	20.0	(11.A)
11.0.C.-	Permanent Access Roads and Parking:						
11.0.C.B	Site Work						
	Road Surfacing						
	Stabilized Aggregate	25,000 Tons	16.00	400,000	60,000	15.0	(11.A)
11.0.1.-	Levees:						
11.0.1.B	Site Work						
	Clearing and Grubbing	38 AC	1,100.00	41,800	8,400	20.1	(11.A)
	Excavation and Embankment						
	Stripping (haul 7 mi)	40,000 CY	3.70	148,000	22,200	15.0	(11.A)
	Excavation (Inspection Trench)	14,000 CY	1.20	16,800	2,500	14.9	(11.A)
	Backfill (Inspection Trench)	14,000 CY	0.80	11,200	1,700	15.2	(11.A)
	Embankment Shaping (Excavate and Recompact on Site)	140,000 CY	1.30	182,000	27,300	15.0	(11.A)
	Embankment Fill (Excavate at borrow, 7 mile haul)	200,000 CY	4.20	840,000	126,000	15.0	(11.A)
	Slope Treatment						
	Seeding, Fertilizing, & Mulching	55 AC	1,200.00	66,000	9,900	15.0	(11.A)
11.0.2.-	Floodwalls: (Stop Log Structure)						
11.0.2.C	Concrete						
	Concrete in Place Including Cement						
	Footings	90 CY	160.00	14,400	2,200	15.3	(11.A)
	Wall	50 CY	230.00	11,500	1,700	14.8	(11.A)
	Reinforcing	21,000 Lbs	0.50	10,500	1,600	15.2	(11.A)
	Stop Logs	150 BF	6.00	900	200	22.2	(11.A)
	Subtotal, Construction Costs:		$	1,823,100			
	Contingency rounded to an average of	15.2 % +/- *		$	276,900		
	NATOMAS AREA LEVEES		**TOTAL:**	$	**2,100,000**		
	PLEASANT GROVE CREEK DETENTION BASIN						
11.0.A.-	Mob., Demob. and Preparatory Work:	1 LS	120,000.00	120,000	24,000	20.0	(11.A)
11.0.C.-	Permanent Access Roads and Parking:						
11.0.C.B	Site Work						
	Road Surfacing						
	Stabilized Aggregate	10,000 Tons	16.00	160,000	24,000	15.0	(11.A)

11.0.1.-	Levees:						
11.0.1.B	Site Work						
	Clearing and Grubbing	30	AC	1,100.00	33,000	6,600	20.0 (11.A)
	Excavation and Embankment						
	Stripping (haul 7 mi)	24,000	CY	3.70	88,800	13,300	15.0 (11.A)
	Excavation (Inspection Trench)	24,000	CY	1.20	28,800	4,300	14.9 (11.A)
	Backfill (Inspection Trench)	24,000	CY	0.80	19,200	2,900	15.1 (11.A)
	Embankment Fill	410,000	CY	4.20	1,722,000	258,300	15.0 (11.A)
	(Excavate at borrow, 7 mile haul)						
	Slope Treatment						
	Seeding, Fertilizing, & Mulching	40	AC	1,200.00	48,000	7,200	15.0 (11.A)
11.0.G.-	Drainage						
11.0.G.B	Sitework						
	Excavation						
	Remove, Stockpile, & Replace						
	Existing Levee	3,000	CY	1.30	3,900	800	20.5 (11.A)
	Structural Fill	830	CY	8.00	6,640	1,300	19.6 (11.A)
	Riprap	270	Tons	23.00	6,210	900	14.5 (11.A)
	48" RCP	100	LF	90.00	9,000	1,400	15.6 (11.A)
11.1.G.C	Concrete						
	Concrete, in Place						
	Concrete Box Culvert	900	CY	300.00	270,000	40,500	15.0 (11.A)
	Reinforcing Steel	79,000	Lbs	0.50	39,500	5,900	14.9 (11.A)
11.1.G.E	Metals						
	Slide Gate w/frame (8'x8')	6	EA	16,000.00	96,000	14,400	15.0 (11.A)
	Slide Gate w/frame (4'x4')	1	EA	4,000.00	4,000	600	15.0 (11.A)
	Subtotal, Construction Costs:			$	2,655,050		
	Contingency rounded to an average of	16.8 % +/- *			$	444,950	
	PLEASANT GROVE CREEK DETENTION BASIN		**TOTAL:**		$	3,100,000	
	Subtotal, Construction Costs:			$	4,478,150		
	Contingency rounded to an average of	16.1 % +/- *			$	721,850	
11.-.-.-	**LEVEES AND FLOODWALLS**		**TOTAL:**		$	5,200,000	
13.-.-.-	**PUMPING PLANT**						
13.0.A.-	Mob., Demob. and Preparatory Work:	1	LS	120,000.00	120,000	24,000	20.0 (13.A)
13.0.B.-	Care and Diversion of Water:						
13.0.B.B	Site Work						
	Diversion Pipe	600	LF	75.00	45,000	6,750	15.0 (13.A)
13.0.B.Q	Mechanical						
	Unwatering Pumps	4	Each	4,000.00	16,000	2,400	15.0 (13.A)
13.0.C.-	Permanent Access Roads and Parking						
13.0.C.B	Site Work						
	Clearing and Grubbing	0.75	AC	1,100.00	825	200	24.2 (13.A)
	Embankment	11,400	CY	4.60	52,440	7,900	15.1 (13.A)
	Topsoil	270	CY	10.00	2,700	400	14.8 (13.A)
	Grass Sedding	0.50	AC	1,200.00	600	90	15.0 (13.A)
13.0.D.-	Earthwork for Structures:						
13.0.D.B	Site Work						
	Clearing and Grubbing	0.50	AC	1,100.00	550	100	18.2 (13.A)
	Excavation, Common	4,350	CY	5.20	22,620	4,500	19.9 (13.A)
	Structural Mat	3,465	CY	5.20	18,018	3,600	20.0 (13.A)

DETAILED ESTIMATE OF FIRST COST TABLE 2 A4

Natomas Levees and Pump Station

13.0.1.-	Pumping Plant Substructure:					
13.0.1.C	Concrete (Including cement, reinforcing, and waterstops)					
	Slab on Grade	1,315 CY	110.00	144,650	21,700	15.0 (13.A)
	Walls	1,465 CY	330.00	483,450	72,500	15.0 (13.A)
	Footings	105 CY	290.00	30,450	4,600	15.1 (13.A)
	Retaining Walls	230 CY	330.00	75,900	11,400	15.0 (13.A)
	Floor Slab	450 CY	120.00	54,000	8,100	15.0 (13.A)
	Transformer Containment	30 CY	110.00	3,300	500	15.2 (13.A)
	Fuel Farm Containment	55 CY	110.00	6,050	900	14.9 (13.A)
13.0.1.E	Metals					
	Grating and Trash Racks	1 LS	500,000.00	500,000	75,000	15.0 (13.A)
13.0.2.E	Metals					
	Pre-engineered Steel Building	624 SF	50.00	31,200	4,700	15.1 (13.A)
13.0.3.-	Canals:					
13.0.3.B	Site Work					
	Clearing and Debris Removal	1.25 AC	1,100.00	1,375	280	20.4 (13.A)
	Excavation, Common	2,600 CY	3.40	8,840	1,300	14.7 (13.A)
	Slope Treatment					
	Gravel Bedding	690 TON	16.00	11,040	1,700	15.4 (13.A)
	Riprap	630 CY	23.00	14,490	2,200	15.2 (13.A)
	Rockfill Slope Protection	370 CY	23.00	8,510	1,300	15.3 (13.A)
	Topsoil	140 CY	10.00	1,400	200	14.3 (13.A)
	Grass Seeding	0.25 AC	1,200.00	300	50	16.7 (13.A)
13.0.3.-	Pumping Machinery and Appurtenances:					
13.0.3.Q	Mechanical					
	Diesel Engines and Pumps	1 LS	700,000.00	700,000	105,000	15.0 (13.A)
13.0.3.R	Electrical					
	Main Pump Electric Motors and Motor Control Center	1 LS	270,000.00	270,000	40,500	15.0 (13.A)
13.0.4.-	Gates and Valves:					
13.0.4.Q	Mechanical					
	Sluice and Flap Gates	1 LS	400,000.00	400,000	60,000	15.0 (13.A)
13.0.5.-	Auxiliary Equipment:					
13.0.5.Q	Mechanical					
	Testing, Startup, and Checkout Fuel Storage and Piping	1 LS	60,000.00	60,000	9,000	15.0 (13.A)
13.0.5.R	Electrical					
	Emergency Generator, Switchgear and Buswork, Lighting, Grounding, Conduit and Wire, and Misc Electrical Work and Testing	1 LS	625,000.00	625,000	90,000	14.4 (13.A)
13.0.6.-	Utilities:					
13.0.6.R	Electrical					
	Telephone	1 LS	800.00	800	120	15.0 (13.A)
13.0.R.-	Associated General Items					
13.0.R.B	Site Work					
	Staff Gages	1 Each	300.00	300	50	16.7 (13.A)
	Fences	770 LF	11.00	8,470	1,000	11.8 (13.A)
	Gates	2 Each	600.00	1,200	200	16.7 (13.A)
	Small Crane	1 Each	17,000.00	17,000	2,600	15.3 (13.A)
	Fire Extinguishers	5 Each	100.00	500	100	20.0 (13.A)
	Subtotal, Construction Costs:		$	3,736,978		
	Contingency rounded to an average of	15.1 % +/- *			$ 563,022	
13.-.-.-	PUMPING PLANT		TOTAL:		$ 4,300,000	

NON-FEDERAL

01.-.-.-	**LANDS AND DAMAGES**				
01.B.-.-	POST-AUTHORIZATION PLANNING				
01.B.1.-	Develop Cost Estimate	81 MD	450.00	36,450	
01.B.2.-	Develop Acquisiton Schedule	75 MD	450.00	33,750	
01.B.9.-	Contingencies				7,000
01.D.-.-	ACQUISITIONS				
01.D.1.-	Attorney's Opinion				
01.D.1.E	Review of Documents	41 MD	450.00	18,450	
01.D.1.Z	All Other	80 MD	450.00	36,000	
01.D.2.-	Mapping, Survey and Tract Ownership				
01.D.2.D	Prepare Documents	622 MD	450.00	279,900	
01.D.2.E	Review of Documents	467 MD	450.00	210,150	
01.D.2.F	Review for Compliance	233 MD	450.00	104,850	
01.D.2.Z	All Other	233 MD	450.00	104,850	
01.D.3.-	Title Evidence				
01.D.3.D	Prepare Documents	208 MD	450.00	93,600	
01.D.3.Z	All Other	103 MD	450.00	46,350	
01.D.4.-	Negotiations and Closing				
01.D.4.D	Prepare Documents	1,209 MD	450.00	544,050	
01.D.4.E	Review of Documents	493 MD	450.00	221,850	
01.D.4.F	Review for Compliance	246 MD	450.00	110,700	
01.D.4.Z	All Other	291 MD	450.00	130,950	
01.D.5.-	Condemnation (Pre-DT Filing)				
01.D.5.D	Prepare Documents	109 MD	450.00	49,050	
01.D.5.E	Review of Documents	41 MD	450.00	18,450	
01.D.5.F	Review for Compliance	21 MD	450.00	9,450	
01.D.5.Z	All Other	21 MD	450.00	9,450	
01.D.9.-	Contingencies				198,600
01.E.-.-	CONDEMNATION (POST-DT FILING)				
01.E.0.D	Prepare Documents	83 MD	450.00	37,350	
01.E.0.E	Review of Documents	21 MD	450.00	9,450	
01.E.0.F	Review for Compliance	20 MD	450.00	9,000	
01.E.0.Z	All Other	1 LS		168,000	
01.E.9.-	Contingencies				22,400
01.F.-.-	APPRAISALS				
01.F.1.-	Staff Appraisals				
01.F.1.H	Prepare Documents	778 MD	450.00	350,100	
01.F.1.J	Review of Documents	311 MD	450.00	139,950	
01.F.1.F	Review for Compliance	311 MD	450.00	139,950	
01.F.1.Z	All Other	156 MD	450.00	70,200	
01.F.2.-	Contract Appraisals				
01.F.2.H	Prepare Documents	93 MD	450.00	41,850	
01.F.2.J	Review of Documents	93 MD	450.00	41,850	
01.F.9.-	Contingencies				78,400
01.H.-.-	RELOCATION ASSISTANCE				
01.H.1.-	PL 91-646				
01.H.1.D	Prepare Documents	40 MD	450.00	18,000	
01.H.1.E	Review of Documents	10 MD	450.00	4,500	
01.H.9.-	Contingencies				5,600
01.K.-.-	TEMPORARY PERMITS				
01.K.0.D	Prepare Documents	83 MD	450.00	37,350	
01.K.0.Z	All Other	41 MD	450.00	18,450	
01.K.9.-	Contingencies				14,000
01.M.-.-	REAL ESTATE RECEIPTS/PAYMENTS				
01.M.3.-	Land Payments	1 LS		7,470,000	
01.M.4.-	Relocation Assistance Payments (PL 91-646)	1 LS		180,000	

DETAILED ESTIMATE OF FIRST COST TABLE 2 A6

Natomas Levees and Pump Station

01.M.5.-	Damage Payments	1 LS		747,000			
01.M.9.-	Contingencies				1,871,300		
	Subtotal, Construction Costs:			$ 11,541,300			
	Contingency rounded to an average of	18.7 % +/- *			$ 2,158,700		(1.B)
01.-.-.-	**LANDS AND DAMAGES**		**TOTAL:**		**$ 13,700,000**		
02.-.-.-	**RELOCATIONS**						
02.1.-.-	**ROADS, Construction Activities**						
	MAIN AVENUE REPLACEMENT						
02.1.1.-	Care of Traffic:						
02.1.1.B	Site Work						
	Temporary Detour Roads	1 LS	50,000.00	50,000	10,000	20.0	(2.A)
02.1.2.-	Construct Roadbed to Subgrade						
02.1.2.B	Site Work						
	Site Preparation:						
	Remove Existing Structure	1 LS	50,000.00	50,000	10,000	20.0	(2.A)
	Excavation: Stripping	1,200 CY	1.80	2,160	300	13.9	(2.A)
	Embankment:						
	Random Fill (8 mi haul)	40,000 CY	4.80	192,000	28,800	15.0	(2.A)
02.1.2.C	Concrete						
	Earth Retaining Wall Panels	700 CY	300.00	210,000	32,000	15.2	(2.A)
02.1.3.-	Road Surfacing						
02.1.3.B	Site Work						
	Base Course	900 CY	29.00	26,100	3,900	14.9	(2.A)
	Asphaltic Concrete Pavement	2,400 Tons	46.00	110,400	16,600	15.0	(2.A)
02.1.J.-	Bridges, Foundations:						
02.1.J.B	Site Work						
	Site Preparation:						
	Excavation, Common	300 CY	9.00	2,700	700	25.9	(2.B)
	Structural Fill	120 CY	12.00	1,440	360	25.0	(2.B)
	Concrete Bearing Piling	2,000 LF	30.00	60,000	15,000	25.0	(2.B)
02.1.J.C	Concrete						
	Concrete, in Place						
	Footings, on Grade	100 CY	140.00	14,000	3,500	25.0	(2.B)
	Reinforcing Steel	15,000 Lbs	0.50	7,500	1,900	25.3	(2.B)
02.1.K.-	Bridges, Abutments and Piers:						
02.1.K.C	Concrete						
	Concrete, in Place						
	Abutments	100 CY	170.00	17,000	4,000	23.5	(2.B)
	Piers	260 CY	370.00	96,200	24,000	24.9	(2.B)
	Reinforcing Steel	27,000 Lbs	0.50	13,500	3,000	22.2	(2.B)
	Cement	1,500 Cwt	4.00	6,000	1,500	25.0	(2.B)
02.1.L.-	Bridge, Superstructure and Deck:						
02.1.L.C	Concrete						
	Concrete, in Place						
	Deck	3,900 CY	390.00	1,521,000	228,000	15.0	(2.A)
	Rails and Curbs	100 CY	490.00	49,000	7,000	14.3	(2.A)
	Reinforcing Steel	600,000 Lbs	0.50	300,000	45,000	15.0	(2.A)
	Cement	30,000 Cwt	4.00	120,000	18,000	15.0	(2.A)
02.1.L.E	Metals						
	Railing	1,400 LF	23.00	32,200	4,800	14.9	(2.A)

	ASCOT AVENUE RAMP						
02.1.1.-	Care of Traffic:						
02.1.1.B	Site Work						
	Temporary Detour	1 LS	5,000.00	5,000	1,000	20.0	(2.A)
02.1.2.-	Construct Roadbed to Subgrade						
02.1.2.B	Site Work						
	Excavation						
	Remove Existing Asphalt	930 SY	5.00	4,650	700	15.1	(2.A)
	Remove E. Base Course (Stockpile)	110 CY	12.00	1,320	200	15.2	(2.A)
	Embankment						
	Random Fill	900 CY	4.40	3,960	600	15.2	(2.A)
02.1.3.-	Road Surfacing						
02.1.3.B	Site Work						
	Base Course 4" SABC	110 CY	29.00	3,190	500	15.7	(2.A)
	Asphaltic Conc Pvmnt 2" (940 SY)	100 Tons	46.00	4,600	700	15.2	(2.A)
	LEVEE ROAD REPLACEMENT STA 607+40 TO 639+00 (South of Sankey Road)						
02.1.1.-	Care of Traffic:						
02.1.1.B	Site Work						
	Temporary Detour	1 LS	7,000.00	7,000	1,000	14.3	(2.A)
02.1.2.-	Construct Roadbed to Subgrade						
02.1.2.B	Site Work						
	Excavation						
	Remove Existing Asphalt	7,000 SY	5.00	35,000	5,000	14.3	(2.A)
	Remove E. Base Course (Stockpile)	800 CY	12.00	9,600	1,000	10.4	(2.A)
02.1.3.-	Road Surfacing						
02.1.3.B	Site Work						
	Base Course 4" SABC	1,100 CY	29.00	31,900	4,800	15.0	(2.A)
	Asphaltic Conc Pvmnt 2" (9800 SY)	1,100 Tons	46.00	50,600	7,600	15.0	(2.A)
	SANKEY ROAD RAMP						
02.1.1.-	Care of Traffic:						
02.1.1.B	Site Work						
	Temporary Detour Roads	1 LS	5,000.00	5,000	1,000	20.0	(2.A)
02.1.2.-	Construct Roadbed to Subgrade						
02.1.2.B	Site Work						
	Excavation						
	Remove Existing Asphalt	350 SY	5.00	1,750	300	17.1	(2.A)
	Remove E. Base Course (Stockpile)	50 CY	12.00	600	90	15.0	(2.A)
	Embankment						
	Random Fill	350 CY	4.00	1,400	200	14.3	(2.A)
02.1.3.-	Road Surfacing						
02.1.3.B	Site Work						
	Base Course 4" SABC	50 CY	29.00	1,450	200	13.8	(2.A)
	Asphaltic Conc Pvmnt 2" (380 SY)	40 Tons	46.00	1,840	300	16.3	(2.A)
	FIFIELD ROAD RAMP						
02.1.1.-	Care of Traffic:						
02.1.1.B	Site Work						
	Temporary Detour Roads	1 LS	5,000.00	5,000	1,000	20.0	(2.A)
02.1.2.-	Construct Roadbed to Subgrade						
02.1.2.B	Site Work						
	Excavation						
	Remove Existing Asphalt	19,200 SY	5.00	96,000	14,000	14.6	(2.A)
	Remove E. Base Course (Stockpile)	2,180 CY	12.00	26,160	3,900	14.9	(2.A)
02.1.3.-	Road Surfacing						
02.1.3.B	Site Work						
	Base Course 4" SABC	2,400 CY	29.00	69,600	10,000	14.4	(2.A)
	Asphaltic Conc Pvmnt 2"(22,000 SY)	2,400 Tons	46.00	110,400	16,600	15.0	(2.A)

DETAILED ESTIMATE OF FIRST COST TABLE 2 A8

Natomas Levees and Pump Station

	HOWSLEY RD RAMP						
02.1.1.-	Care of Traffic:						
02.1.1.B	Site Work						
	Temporary Detour Roads	1 LS	5,000.00	5,000	1,000	20.0	(2.A)
02.1.2.-	Construct Roadbed to Subgrade						
02.1.2.B	Site Work						
	Excavation						
	Remove Existing Asphalt	500 SY	5.00	2,500	400	16.0	(2.A)
	Remove E. Base Course (Stockpile)	60 CY	12.00	720	100	13.9	(2.A)
	Embankment: Random Fill	500 CY	4.00	2,000	300	15.0	(2.A)
02.1.3.-	Road Surfacing						
02.1.3.B	Site Work						
	Base Course 4" SABC	80 CY	29.00	2,320	300	12.9	(2.A)
	Asphaltic Conc Pvmnt 2" (720 SY)	80 Tons	46.00	3,680	600	16.3	(2.A)
	PLEASANT GROVE DETENTION BASIN						
02.1.1.-	Care of Traffic:						
02.1.1.B	Site Work						
	Temporary Detour Roads	1 LS	2,000.00	2,000	300	15.0	(2.A)
02.1.2.-	Construct Roadbed to Subgrade						
02.1.2.B	Site Work						
	Excavation						
	Remove Existing Asphalt	400 SY	5.00	2,000	300	15.0	(2.A)
	Remove E. Base Course (Stockpile)	50 CY	12.00	600	100	16.7	(2.A)
02.1.3.-	Road Surfacing						
02.1.3.B	Site Work						
	Base Course 4" SABC	50 CY	29.00	1,450	200	13.8	(2.A)
	Asphaltic Conc Pvmnt 2" (450 SY)	50 Tons	46.00	2,300	300	13.0	(2.A)
	Subtotal, Construction Costs:		$	3,381,790			
	Contingency rounded to an average of	15.3 % +/- *		$	518,210		
02.1.-.-	**ROADS, Construction Activities**	**TOTAL:**		**$**	**3,900,000**		
02.2.-.-	**RAILROADS, Construction Activities**						
02.2.1.-	Care of Traffic:						
02.2.1.B	Site Work						
	Temporary Detour Track	500 LF	40.00	20,000	4,000	20.0	(2.A)
02.2.3.-	Track Work:						
02.2.3.B	Site Work						
	Remove, Salvage and Reuse						
	Rails and Accessories	150 LF	30.00	4,500	900	20.0	(2.A)
	Ballast	100 CY	30.00	3,000	600	20.0	(2.A)
	Ties	150 Each	35.00	5,250	1,100	21.0	(2.A)
	Subtotal, Construction Costs:		$	32,750			
	Contingency rounded to an average of	22.1 % +/- *		$	7,250		
02.2.-.-	**RAILROADS, Construction Activities**	**TOTAL:**		**$**	**40,000**		
02.3.-.-	**CEMETERIES, UTILITIES, AND STRUCTURES Construction Activities**						
02.3.2.-	Utilities						
02.3.2.B	Site Work						
	Remove and Replace						
	Chain Link Fence	2,700 LF	12.00	32,400	4,900	15.1	(2.A)
	Barbed Wire Fence	1,200 LF	3.20	3,840	600	15.6	(2.A)

02.3.2.R	Electrical Remove and Replace Primary Power Line	1,800 LF	12.00	21,600	3,200	14.8	(2.A)
	Subtotal, Construction Costs:		$	57,840			
	Contingency rounded to an average of	21.0 % +/- *		$	12,160		
02.3.-.-	**CEMETERIES, UTILITIES, AND STRUCTURES**		**TOTAL:**	**$**	**70,000**		
	FEDERAL AND NON-FEDERAL						
30.-.-.-	**PLANNING, ENGINEERING & DESIGN**						
30.C.-.-	LOCAL COOPERATIVE AGREEMENTS						
30.C.A.-	Draft LCA	10 DAYS	560.00	5,600			
30.C.B.-	Final LCA & Financial Plan	2 MM	560.00	1,120			
30.C.1.-	LCA Negotiations	2 MM	560.00	1,120			
30.C.Z.-	Contingencies	1 JOB LS			2,500		
30.D.-.-	ENVIRONMENTAL AND REGULATORY ACTIVITIES						
30.D.C.-	Supplemental EIS	400 DAYS	300.00	120,000			
30.D.2.-	401, 404, & ROD	90 DAYS	300.00	27,000			
30.D.Z.-	Contingencies	1 JOB LS			14,600		
30.E.-.-	DESIGN RELATED ENGINEERING						
30.E.1.-	Subsurface Explorations	1 JOB LS		50,000			
30.E.2.-	Sampling, Testing, & Analysis	1 JOB LS		60,000			
30.E.Z.-	Contingencies	1 JOB LS			11,000		
30.F.-.-	LETTER REPORT						
30.F.A.-	Draft Letter Report	100 DAYS	310.00	31,000			
30.F.B.-	Final Letter Report	30 DAYS	310.00	9,300			
30.F.Z.-	Contingencies	1 JOB LS			9,900		
30.G.-.-	FDM (LEVEE)						
30.G.A.-	Draft Feature Design Memorandum	3490 DAYS	310.00	1,081,900			
30.G.B.-	Final FDM	870 DAYS	310.00	269,700			
30.G.D.-	AE Contract Administration	130 DAYS	310.00	40,300			
30.G.F.-	Value Engineering Studies	190 DAYS	310.00	58,900			
30.G.Z.-	Contingencies	1 JOB LS			145,100		
30.G.-.-	FDM (MITIGATION)						
30.G.A.-	Draft Feature Design Memorandum	100 DAYS	310.00	31,000			
30.G.B.-	Final FDM	30 DAYS	310.00	9,300			
30.G.Z.-	Contingencies	1 JOB LS			9,900		
30.H.-.-	PLANS AND SPECIFICATIONS-LEVEE						
30.H.A.-	Preliminary Design	960 DAYS	310.00	297,600			
30.H.B.-	Final Design	270 DAYS	310.00	83,700			
30.H.C.-	Design Revisions	140 DAYS	310.00	43,400			
30.H.D.-	AE Contract Administration	70 DAYS	310.00	21,700			
30.H.E.-	BCO Review	90 DAYS	310.00	27,900			
30.H.Z.-	Contingencies	1 JOB LS			47,300		
30.H.-.-	PLANS AND SPECIFICATIONS-MITIGATION						
30.H.A.-	Preliminary Design	320 DAYS	310.00	99,200			
30.H.B.-	Final Design	60 DAYS	310.00	18,600			
30.H.C.-	Design Revisions	30 DAYS	310.00	9,300			
30.H.E.-	BCO Review	30 DAYS	310.00	9,300			
30.H.Z.-	Contingencies	1 JOB LS			14,000		
30.J.-.-	ENGINEERING DURING CONSTRUCTION						
30.J.H.-	Value Engineering Change Proposals	100 DAYS	310.00	31,000			
30.J.1.-	Review E&D Effort by Constr Contrct	50 DAYS	310.00	15,500			
30.J.2.-	Periodic Inspections	150 DAYS	310.00	46,500			
30.J.9.-	All Engineering During Constr	190 DAYS	310.00	58,900			
30.J.Z.-	Contingencies	1 JOB LS			15,000		

DETAILED ESTIMATE OF FIRST COST TABLE 2 A10

Natomas Levees and Pump Station

30.M.-.-	COST ENGINEERING	260 DAYS	310.00	80,600		
30.P.-.-	PROJECT MANAGEMENT	910 MM	400.00	364,000		
30.P.Z.-	Contingencies	1 JOB LS			36,200	
30.Z.-.-	MISCELLANEOUS ACTIVITIES					
30.Z.1.-	FWS Support	1 JOB LS		90,000		
30.Z.1.-	Surveys (Topographical)	1 JOB LS		350,000		
30.Z.1.-	Surveys (Cultural)	1 JOB LS		100,000		
30.Z.Z.-	Contingencies:	1 JOB LS			50,000	
	Subtotal, Construction Costs:		$	3,543,440		
	Contingency rounded to an average of	10.1 % +/- *		$	356,560	(30.A)
30.-.-.-	**PLANNING, ENGINEERING & DESIGN**		**Subtotal: $**	**$**	**3,900,000**	
	FEDERAL		Subtotal: $	2,837,568		
	Contingency rounded to an average of	9.2 % +/- *			262,432	
	Federal PED			$	3,100,000	
	NON-FEDERAL		Subtotal: $	705,872		
	Contingency rounded to an average of	13.3 % +/- *			94,128	
	Non-Federal PED			$	800,000	
31.-.-.-	**CONSTRUCTION MANAGEMENT (S & I)**					
31.B.-.-	CONTRACT ADMINISTRATION					
31.B.1.-	Pre-award Activities					
31.B.1.1	Resident Office	138 MH	45.00	6,210	1,000	
31.B.1.2	District Office	343 MH	60.00	20,580	2,300	
31.B.2.-	Award Activities	86 MH	60.00	5,160	600	
31.B.3.-	Review/Approval of Contract Payment	828 MH	45.00	37,260	4,100	
31.B.4.-	Contract Modifications					
31.B.4.1	Resident Office	5,519 MH	45.00	248,355	30,000	
31.B.4.2	District Office	429 MH	60.00	25,740	4,000	
31.B.5.-	Progress and Completion Reports	552 MH	45.00	24,840	4,000	
31.D.-.-	REVIEW OF SHOP DRAWINGS					
31.D.1	Resident Office	2,760 MH	45.00	124,200	14,000	
31.D.2	District Office	429 MH	60.00	25,740	3,000	
31.E.-.-	INSPECTION AND QUALITY ASSURANCE					
31.E.1.-	Schedule Compliance	552 MH	45.00	24,840	3,000	
31.E.2.-	Compliance Sampling & Testing					
31.E.2.1	Resident Office	2,070 MH	45.00	93,150	10,300	
31.E.2.2	Laboratory Charges	1 JOB LS		93,140	10,300	
31.E.9.-	Quality Assurance Personnel	7,037 MH	45.00	316,665	40,000	
31.F.-.-	PROJECT OFFICE OPERATION					
31.F.1.-	Resident Office	552 MH	45.00	24,840	4,000	
31.F.2.-	Vehicles and Equipment	1 JOB LS		248,521	30,000	
31.H.-.-	CONTRACTOR CLAIMS & LITIGATIONS	1,502 MH	60.00	90,120	13,500	
31.P.-.-	PROJECT MANAGEMENT	1,502 MH	60.00	90,120	13,500	
	Subtotal, Construction Costs:		$	1,499,481		
	Contingency rounded to an average of	13.4 % +/- *		$	200,519	(31.A)
31.-.-.-	**CONSTRUCTION MANAGEMENT (S & I)**		**TOTAL:**	**$**	**1,700,000**	
	FEDERAL		Subtotal: $	1,200,777		
	Contingency rounded to an average of	13.3 % +/- *			159,223	
	Federal (S&I)				1,360,000	
	NON-FEDERAL		Subtotal: $	298,704		
	Contingency rounded to an average of	13.8 % +/- *			41,296	
	Non-Federal (S&I)				340,000	

NATOMAS RECREATION CONTRACT

COST ESTIMATE
DETAILED ESTIMATE OF FIRST COST   TABLE 2   E1

ACCOUNT NUMBER	ITEM	QUANTITY	UNIT	UNIT PRICE $	AMOUNT $	CONTINGENCY $ *	CONTINGENCY % *	REASON
	Effective Price Date(EPD) 1-Oct-91 Natomas Recreation					* (Figures Rounded)		
	FEDERAL							
01.-.-.-	**LANDS AND DAMAGES**							
01.B.-.-	POST-AUTHORIZATION PLANNING							
01.B.1.-	Develop Cost Estimate	100	MD	430.00	43,000			
01.B.3.-	Real estate design Memorandum	15	MD	450.00	6,750			
01.B.4.-	Evaluate Sponsor Capability	3	MD	530.00	1,590			
01.B.8.-	All Other	168	MD	430.00	72,240			
01.B.9.-	Contingencies					18,700		
01.C.-.-	LOCAL COOPERATION AGREEMENT							
01.C.2.-	Final LCA	4	MD	500.00	2,000			
01.C.9.-	Contingencies					300		
01.D.-.-	ACQUISITIONS							
01.D.1.-	Attorney's Opinion							
01.D.1.E	Review of Documents	4	MD	530.00	2,120			
01.D.2.-	Mapping, Survey and Tract Ownership							
01.D.2.D	Prepare Documents	10	MD	350.00	3,500			
01.D.2.F	Review for Compliance	7	MD	360.00	2,520			
01.D.2.Z	All Other	33	MD	350.00	11,550			
01.D.4.-	Negotiations and Closing							
01.D.4.F	Review for Compliance	50	MD	390.00	19,500			
01.D.4.Z	All Other	73	MD	460.00	33,580			
01.D.9.-	Contingencies					10,900		
01.F.-.-	APPRAISALS							
01.F.1.-	Staff Appraisals							
01.F.1.Z	All Other	2	MD	500.00	1,000			
01.F.2.-	Contract Appraisals							
01.F.2.Z	All Other	220	MD	500.00	110,000			
01.F.9.-	Contingencies					16,700		
	Subtotal, Construction Costs:			$	309,350			
	Contingencies @ average of 16.4 % +/- *				$	50,650		(1.A)
01.-.-.-	**LANDS AND DAMAGES**			**TOTAL:**	$	**360,000**		
14.-.-.-	**RECREATION FACILITIES**							
14.0.A.-	Mob., Demob. and Preparatory Work:	1	LS	80,000.00	80,000	12,000		
14.0.4.-	Day Use Areas:							
14.0.4.B	Site Work							
	Clearing and Grubbing	25	Acre	2,000.00	50,000	7,500		
	Stripping -6"	20,000	CY	2.00	40,000	6,000		
	Embankment	15,000	CY	4.50	67,500	10,000		
	Riprap	2,500	Tons	26.00	65,000	9,800		
	Stabilized Aggregate Base -4"	25,000	Tons	16.00	400,000	60,000		
	Asphaltic Concrete -2"	9,000	Tons	48.00	432,000	65,000		
	Tack Coat	75,000	SY	0.40	30,000	4,500		
	Striping -4" Stripe	60,000	LF	0.20	12,000	1,800		
	Disc (Remove rocks, metals, and Debris)	120,000	SF	0.50	60,000	10,000		
	Gates -12' Metal Pole	10	Each	420.00	4,200	630		
	Trail Markers	75	Each	65.00	4,875	731		
	Subtotal, Construction Costs:			$	1,245,575			
	Contingency rounded to an average of 12.4 % +/- *				$	154,425		(14.A)
14.-.-.-	**RECREATION FACILITIES**			**TOTAL:**	$	**1,400,000**		

DETAILED ESTIMATE OF FIRST COST TABLE 2 E2

Natomas Recreation

30.-.-.-	**PLANNING, ENGINEERING & DESIGN**					
30.C.-.-	LOCAL COOPERATIVE AGREEMENTS					
30.C.A.-	Draft LCA	15 DAYS	560.00	8,400		
30.C.B.-	Final LCA & Financial Plan	5 DAYS	560.00	2,800		
30.C.1.-	LCA Negotiations	5 DAYS	560.00	2,800		
30.C.Z.-	Contingencies	1 JOB LS			3,600	
30.D.-.-	ENVIRONMENTAL AND REGULATORY ACTIVITIES					
30.D.C.-	Supplemental EIS	40 DAYS	300.00	12,000		
30.D.Z.-	Contingencies	1 JOB LS			3,300	
30.F.-.-	LETTER REPORT					
30.F.A.-	Draft Letter Report	60 DAYS	310.00	18,600		
30.F.B.-	Final Letter Report	10 DAYS	310.00	3,100		
30.F.Z.-	Contingencies	1 JOB LS			5,600	
30.F.-.-	FEATURE DESIGN MEMORANDUM (LEVEES)					
30.F.A.-	Draft FDM	180 DAYS	310.00	55,800		
30.F.B.-	Final FDM	50 DAYS	310.00	15,500		
30.F.Z.-	Contingencies	1 JOB LS			7,100	
30.F.-.-	FEATURE DESIGN MEMORANDUM (MITIGATION)					
30.F.A.-	Draft FDM	10 DAYS	310.00	3,100		
30.F.B.-	Final FDM	3 DAYS	310.00	930		
30.F.Z.-	Contingencies	1 JOB LS			1,000	
30.H.-.-	PLANS AND SPECIFICATIONS					
30.H.A.-	Preliminary Design	190 DAYS	310.00	58,900		
30.H.B.-	Final Design	20 DAYS	310.00	6,200		
30.H.C.-	Design Revisions	20 DAYS	310.00	6,200		
30.H.E.-	BCO Review	20 DAYS	310.00	6,200		
30.H.Z.-	Contingencies	1 JOB LS			8,000	
30.J.-.-	ENGINEERING DURING CONSTRUCTION					
30.J.2.-	Periodic Inspections	10 DAYS	310.00	3,100		
30.J.9.-	All Engineering During Constr	20 DAYS	310.00	6,200		
30.J.Z.-	Contingencies	1 JOB LS			2,600	
30.M.-.-	COST ENGINEERING	40 DAYS	310.00	12,400		
30.P.-.-	PROJECT MANAGEMENT	110 DAYS	400.00	44,000		
30.P.Z.-	Contingencies	1 JOB LS			4,300	
30.Z.-.-	MISCELLANEOUS ACTIVITIES					
30.Z.1.-	FWS Support	1 JOB LS		11,000		
30.Z.1.-	Surveys (Topographical)	1 JOB LS		19,800		
30.Z.1.-	Surveys (Cultural)	1 JOB LS		10,000		
	Subtotal, Construction Costs:		$	307,030		
	Contingency rounded to an average of	10.7 % +/- *		$	32,970	(30.A)
30.-.-.-	**PLANNING, ENGINEERING & DESIGN**	**Subtotal:**		**$**	**340,000**	
31.-.-.-	**CONSTRUCTION MANAGEMENT (S & I)**					
31.B.-.-	CONTRACT ADMINISTRATION					
31.B.1.-	Pre-award Activities					
31.B.1.1	Resident Office	11 MH	45.00	495	50	
31.B.1.2	District Office	26 MH	60.00	1,560	170	
31.B.2.-	Award Activities	7 MH	60.00	420	50	
31.B.3.-	Review/Approval of Contract Payment	63 MH	45.00	2,835	300	

DETAILED ESTIMATE OF FIRST COST **TABLE 2** **E3**

Natomas Recreation

31.B.4.-	Contract Modifications					
31.B.4.1	Resident Office	413 MH	45.00	18,585	2,000	
31.B.4.2	District Office	33 MH	60.00	1,980	200	
31.B.5.-	Progress and Completion Reports	42 MH	45.00	1,890	200	
31.D.-.-	REVIEW OF SHOP DRAWINGS					
31.D.1	Resident Office	209 MH	45.00	9,405	1,000	
31.D.2	District Office	33 MH	60.00	1,980	200	
31.E.-.-	INSPECTION AND QUALITY ASSURANCE					
31.E.1.-	Schedule Compliance	42 MH	45.00	1,890	200	
31.E.2.-	Compliance Sampling & Testing					
31.E.2.1	Resident Office	157 MH	45.00	7,065	800	
31.E.2.2	Laboratory Charges	1 JOB LS		42,293	4,700	
31.E.9.-	Quality Assurance Personnel	533 MH	45.00	23,985	2,700	
31.F.-.-	PROJECT OFFICE OPERATION					
31.F.1.-	Resident Office	42 MH	45.00	1,890	200	
31.F.2.-	Vehicles and Equipment	1 JOB LS		112,849	13,000	
31.H.-.-	CONTRACTOR CLAIMS & LITIGATIONS	114 MH	60.00	6,840	800	
31.P.-.-	PROJECT MANAGEMENT	114 MH	60.00	6,840	800	
.Z.Z.-	Contingencies:					
	Subtotal, Construction Costs:		$	242,802		
	Contingency rounded to an average of	11.2 % +/- *		$	27,198	(31.A)
31.-.-.-	**CONSTRUCTION MANAGEMENT (S & I)**		**TOTAL:**	**$**	**270,000**	

NON-FEDERAL

01.-.-.-	**LANDS AND DAMAGES**				
01.C.-.-	LOCAL COOPERATION AGREEMENT				
01.C.1.-	Draft LCA	15 MD	490.00	7,350	
01.C.2.-	Final LCA	3 MD	500.00	1,500	
01.C.9.-	Contingencies				1,300
01.D.-.-	ACQUISITIONS				
01.D.1.-	Attorney's Opinion				
01.D.1.D	Prepare Documents	15 MD	490.00	7,350	
01.D.1.E	Review of Documents	6 MD	480.00	2,880	
01.D.1.Z	All Other	3 MD	500.00	1,500	
01.D.2.-	Mapping, Survey and Tract Ownership				
01.D.2.D	Prepare Documents	103 MD	500.00	51,500	
01.D.2.E	Review of Documents	4 MD	550.00	2,200	
01.D.2.Z	All Other	1 MD	700.00	700	
01.D.3.-	Title Evidence				
01.D.3.D	Prepare Documents	1 LS		131,800	
01.D.3.E	Review of Documents	15 MD	490.00	7,350	
01.D.3.Z	All Other	15 MD	490.00	7,350	
01.D.4.-	Negotiations and Closing				
01.D.4.D	Prepare Documents	15 MD	490.00	7,350	
01.D.4.F	Review for Compliance	3 MD	500.00	1,500	
01.D.4.Z	All Other	15.8 MY	120,000.00	1,896,000	

DETAILED ESTIMATE OF FIRST COST   TABLE 2   E4

Natomas Recreation

01.D.5.-	Condemnation (Pre-DT Filing)					
01.D.5.D	Prepare Documents	15 MD	490.00	7,350		
01.D.5.E	Review of Documents	3 MD	500.00	1,500		
01.D.5.Z	All Other	15 MD	490.00	7,350		
01.D.9.-	Contingencies				321,200	
01.E.-.-	CONDEMNATION (POST-DT FILING)					
01.E.0.D	Prepare Documents	15 MD	490.00	7,350		
01.E.0.E	Review of Documents	3 MD	500.00	1,500		
01.E.0.Z	All Other	73 MD	500.00	36,500		
01.E.9.-	Contingencies				6,800	
01.F.-.-	APPRAISALS					
01.F.1.-	Staff Appraisals					
01.F.1.H	Prepare Documents	11 MY	120,000.00	1,320,000		
01.F.1.J	Review of Documents	58 MD	500.00	29,000		
01.F.1.Z	All Other	29 MD	500.00	14,500		
01.F.2.-	Contract Appraisals					
01.F.2.H	Prepare Documents	1 LS		293,000		
01.F.2.J	Review of Documents	29 MD	500.00	14,500		
01.F.2.Z	All Other	29 MD	500.00	14,500		
01.F.9.-	Contingencies				252,700	
01.K.-.-	TEMPORARY PERMITS					
01.K.0.D	Prepare Documents	6 MD	480.00	2,880		
01.K.0.E	Review of Documents	2 MD	350.00	700		
01.K.0.Z	All Other	29 MD	500.00	14,500		
01.K.9.-	Contingencies				2,800	
01.L.-.-	ENCROACHMENTS AND TRESPASS					
01.L.1.-	Prevention/Detection	15 MD	490.00	7,350		
01.L.2.-	Resolution Activities	6 MD	490.00	2,940		
01.L.9.-	Contingencies				1,600	
01.M.-.-	REAL ESTATE RECEIPTS/PAYMENTS					
01.M.3.-	Land Payments	1 LS		1,326,000		
01.M.5.-	Damage Payments	1 LS		265,000		
01.M.9.-	Contingencies				331,000	
	Subtotal, Construction Costs:			$ 5,492,750		
	Contingencies @ average of 16.5 % +/- *				$ 907,250	(1.B)
01.-.-.-	**LANDS AND DAMAGES**		**TOTAL:**		**$ 6,400,000**	

DAM RELOCATIONS CONTRACT

COST ESTIMATE
DETAILED ESTIMATE OF FIRST COST TABLE 2 D1

ACCOUNT NUMBER	ITEM	QUANTITY	UNIT	UNIT PRICE $	AMOUNT $	CONTINGENCY $ *	CONTINGENCY % *	REASON
	Effective Price Date(EPD) 1-Oct-91 * (Figures Rounded)							
	Highway 49 / Ponderosa Road / Utilities Relocations							
	FEDERAL							
02.A.-.-	**ROADS, Real Estate Activities**							
02.A.B.-	Post-Authorization Planning							
02.A.B.G	Relocation Contract	171	MD	450.00	76,950			
02.A.C.-	Local Cooperation Agreement							
02.A.1.B	Final LCA	4	MD	500.00	2,000			
02.A.D.-	Attorneys' Opinion of Compensability							
02.A.D.Z	All Other	18	MD	530.00	9,540			
02.A.E.-	Mapping, Surveying and Tract Ownership							
02.A.E.D	Prepare Documents	10	MD	350.00	3,500			
02.A.E.F	Review for Compliance	5	MD	360.00	1,800			
02.A.G.-	Negotiations and Closings							
02.A.G.F	Review for Compliance	15	MD	420.00	6,300			
02.A.G.Z	All Other	69	MD	450.00	31,050			
02.A.K.-	Staff Appraisals							
02.A.K.Z	All Other	2	MD	500.00	1,000			
02.A.L.-	Contract Appraisals							
02.A.L.Z	All Other	5	MD	500.00	2,500			
02.A.Z.-	Contingencies:					20,000		
	Subtotal, Construction Costs:			$	134,640			
	Contingencies @ average of 11.4 % +/- *				$	15,360		(1.A)
02.A.-.-	**ROADS, Real Estate Activities**		**TOTAL:**		$	**150,000**		
	NON-FEDERAL							
02.1.-.-	**ROADS, Construction Activities**							
	HIGHWAY 49							
	WARNER RAVINE BRIDGE							
02.1.A.-	Mob., Demob. and Preparatory Work:	1	JOB	LS	820,000	65,600	8.0	
02.1.1.-	Care of Traffic:							
02.1.1.B	Site Work							
	Flagging	1	JOB	LS	185,000	25,900	14.0	
	Temporary Barriers	1	JOB	LS	10,700	1,500	14.0	
02.1.2.-	Construct Roadbed to Subgrade							
02.1.2.B	Site Work							
	Site Preparation:							
	Stripping	1,540	CY	5.20	8,008	1,200	15.0	
	Subgrading	41,600	SF	0.20	8,320	1,200	14.4	
02.1.3.-	Road Surfacing							
02.1.3.B	Site Work							
	Road Surfacing	41,600	SF	1.70	70,720	11,300	16.0	
02.1.J.-	Bridge, Foundations:							
02.1.J.B	Site Work							
	Excavation(open)	9,000	CY	17.00	153,000	38,300	25.0	(2.B)
	Excavation(Shalf W/support)	3,900	CY	60.00	234,000	46,800	20.0	
	Structural Backfill	10,500	CY	21.00	220,500	46,300	21.0	
02.1.J.C	Concrete							
	Concrete	3,900	CY	240.00	936,000	121,700	13.0	
02.1.K.-	Bridge, Abutments and Piers :							
02.1.K.C	Concrete							
	Abutment	1,200	CY	290.00	348,000	34,800	10.0	
	Box Pier	6,000	CY	500.00	3,000,000	360,000	12.0	

DETAILED ESTIMATE OF FIRST COST TABLE 2 D2
Highway 49 / Ponderosa Road / Utilities Relocations

02.1.K.E	Metal						
	Reinforcing Steel	2,100,000 LB	0.50	1,050,000	157,500	15.0	
02.1.L.-	Bridge, Superstructure and Deck:						
02.1.L.C	Concrete						
	Superstructure, Concrete	14,800 CY	630.00	9,324,000	1,118,900	12.0	
	Deck, Concrete	915 CY	370.00	338,550	33,900	10.0	
02.1.L.E	Metal						
	Prestress Steel	2,800,000 LB	1.70	4,760,000	714,000	15.0	
	Reinforcing Steel	3,700,000 LB	0.50	1,850,000	277,500	15.0	
	Railing, Bearing	55,000 LB	5.00	275,000	22,000	8.0	
	Expansion Joint	80 LF	770.00	61,600	3,300	5.4	
02.1.M.-	Bridge, Associated General Items:						
02.1.M.B	Site Work						
	Guardrail	2,080 LF	38.00	79,040	7,900	10.0	
	Striping	8,000 LF	0.60	4,800	500	10.4	
	Subtotal, Construction Costs:		$	23,737,238			
	Contingencies @ average of 12.9 % +/- *			$	3,062,762		(2.A)
02.1.-.-	**WARNER RAVINE BRIDGE**		**TOTAL:**	**$**	**26,800,000**		
	AMERICAN RIVER NORTH FORK BRIDGE						
02.1.A.-	Mob., Demob. and Preparatory Work:	1 JOB	LS	1,100,000	88,000	8.0	
02.1.1.-	Care of Traffic:						
02.1.1.B	Site Work						
	Flagging	1 JOB	LS	230,000	32,200	14.0	
	Temporary Barriers	1 JOB	LS	10,600	1,500	14.2	
02.1.2.-	Construct Roadbed to Subgrade						
02.1.2.B	Site Work						
	Site Preparation:						
	Stripping	178 CY	4.90	872	100	11.5	
	Subgrading	4,800 SF	0.20	960	100	10.4	
02.1.3.-	Road Surfacing						
02.1.3.B	Site Work						
	Road Surfacing	4,800 SF	2.50	12,000	1,900	15.8	
02.1.J.-	Bridge, Foundations:						
02.1.J.B	Site Work						
	Excavation(open)	12,600 CY	17.00	214,200	53,600	25.0	(2.B)
	Excavation(Shalf W/support)	9,800 CY	60.00	588,000	117,600	20.0	
	Structural Backfill	7,000 CY	21.00	147,000	29,400	20.0	
02.1.J.C	Concrete						
	Concrete	9,800 CY	180.00	1,764,000	229,300	13.0	
02.1.K.-	Bridge, Abutments and Piers :						
02.1.K.C	Concrete						
	Abutment	1,200 CY	290.00	348,000	34,800	10.0	
	Box Pier	14,000 CY	500.00	7,000,000	840,000	12.0	
02.1.K.E	Metal						
	Reinforcing Steel	4,800,000 LB	0.50	2,400,000	360,000	15.0	
02.1.L.-	Bridge, Superstructure and Deck:						
02.1.L.C	Concrete						
	Superstructure, Concrete	17,000 CY	560.00	9,520,000	1,142,400	12.0	
	Deck, Concrete	1,050 CY	330.00	346,500	34,700	10.0	
02.1.L.E	Metal						
	Prestress Steel	3,200,000 LB	1.70	5,440,000	816,000	15.0	
	Reinforcing Steel	4,200,000 LB	0.50	2,100,000	315,000	15.0	
	Railing, Bearing	64,000 LB	5.00	320,000	25,600	8.0	
	Expansion Joint	80 LF	770.00	61,600	3,300	5.4	

DETAILED ESTIMATE OF FIRST COST TABLE 2 D3
Highway 49 / Ponderosa Road / Utilities Relocations

02.1.M.-	Bridge, Associated General Items:						
02.1.M.B	Site Work						
	Guardrail	240 LF	38.00	9,120	900	9.9	
	Striping	7,040 LF	0.60	4,224	400	9.5	
	Subtotal, Construction Costs:		$	31,617,076			
	Contingencies @ average of 12.9 % +/- *			$	4,082,924		(2.A)
02.1.-.-	**AMERICAN RIVER NORTH FORK BRIDGE**		**TOTAL:**	**$**	**35,700,000**		
	VIADUCT BRIDGE						
02.1.A.-	Mob., Demob. and Preparatory Work:	1 JOB	LS	425,000	34,000	8.0	
02.1.1.-	Care of Traffic:						
02.1.1.B	Site Work						
	Flagging	1 JOB	LS	100,000	14,000	14.0	
	Temporary Barriers	1 JOB	LS	11,200	1,600	14.3	
02.1.2.-	Construct Roadbed to Subgrade						
02.1.2.B	Site Work						
	Site Preparation:						
	Stripping	550 CY	5.20	2,860	400	14.0	
	Subgrading	14,850 SF	0.20	2,970	400	13.5	
02.1.3.-	Road Surfacing						
02.1.3.B	Site Work						
	Road Surfacing	14,850 SF	2.10	31,185	5,000	16.0	
02.1.J.-	Bridge, Foundations:						
02.1.J.B	Site Work						
	Excavation(open)	1,000 CY	26.00	26,000	6,500	25.0	(2.B)
	Structural Backfill	5,200 CY	21.00	109,200	21,800	20.0	
02.1.J.C	Concrete						
	Concrete	300 CY	380.00	114,000	14,800	13.0	
02.1.K.-	Bridge, Abutments and Piers :						
02.1.K.C	Concrete						
	Abutment	1,600 CY	340.00	544,000	54,400	10.0	
	Box Pier	960 CY	500.00	480,000	57,600	12.0	
	Drill & Grout Piles	2,100 LF	70.00	147,000	22,100	15.0	
	Piles (16")	2,100 LF	50.00	105,000	15,800	15.0	
02.1.K.E	Metal						
	Prestess Steel	30,000 LB	1.70	51,000	7,700	15.1	
	Reinforcing Steel	500,000 LB	0.50	250,000	37,500	15.0	
02.1.L.-	Bridge, Superstructure and Deck:						
02.1.L.C	Concrete						
	Superstructure, Concrete	6,490 CY	540.00	3,504,600	420,600	12.0	
	Deck, Concrete	550 LF	340.00	187,000	24,300	13.0	
02.1.L.E	Metal						
	Prestress Steel	1,000,000 LB	1.70	1,700,000	255,000	15.0	
	Reinforcing Steel	1,200,000 LB	0.50	600,000	90,000	15.0	
	Railing, Bearing	33,500 LB	5.20	174,200	13,900	8.0	
	Expansion Joint	80 LF	770.00	61,600	3,800	6.2	
02.1.M.-	Bridge, Associated General Items:						
02.1.M.B	Site Work						
	Guardrail	743 LF	38.00	28,234	2,800	9.9	
	Striping	4,285 LF	0.60	2,571	300	11.7	
	Subtotal, Construction Costs:		$	8,657,620			
	Contingencies @ average of 13.2 % +/- *			$	1,142,380		(2.A)
02.1.-.-	**VIADUCT BRIDGE**		**TOTAL:**	**$**	**9,800,000**		

DETAILED ESTIMATE OF FIRST COST TABLE 2 D4

Highway 49 / Ponderosa Road / Utilities Relocations

	PONDEROSA ROAD						
02.1.A.-	Mob., Demob. and Preparatory Work:	1 JOB	LS	250,000	27,500	11.0	
02.1.1.-	Care of Traffic:						
02.1.1.B	Site Work						
	Temporary Detour Roads	1 JOB	LS	12,400	1,700	13.7	
	Maintain Detour Roads	1 JOB	LS	14,600	2,000	13.7	
	Flagging	9 MO	14,600	131,400	18,400	14.0	
	Barriers and Markings	1 JOB	LS	4,400	600	13.6	
	Temp Guard Railing	80 LF	25	2,000	300	15.0	
02.1.2.-	Construct Roadbed to Subgrade						
02.1.2.B	Site Work						
	Clearing & Grubbing	4 ACRE	2,500	10,000	1,400	14.0	
	Excavation						
	Stripping	5,000 CY	2.30	11,500	2,000	17.4	
	Common	285,000 CY	3.00	855,000	145,400	17.0	(2.B)
	Rock	25,000 CY	11.00	275,000	46,800	17.0	
	Structure	300 CY	14.00	4,200	700	16.7	
	Embankment						
	Random Fill	285,000 CY	1.60	456,000	77,500	17.0	
	Rock Fill	25,000 CY	7.00	175,000	29,800	17.0	
	Top Soil	5,000 CY	11.00	55,000	9,400	17.1	
	Drainage & Erosion Control						
	Excavation	600 CY	11.00	6,600	900	13.6	
	Filter Material	3,000 CY	19.00	57,000	8,000	14.0	
	Pipe, Culverts						
	24" CMP	460 LF	40.00	18,400	2,600	14.1	
	Backfill	300 CY	9.00	2,700	400	14.8	
	Paved Gutters, V Ditch	3,000 LF	4.00	12,000	1,700	14.2	
	Catch Basin	20 EA	1750.00	35,000	6,000	17.1	
	Rock Slope Protection	80 CY	56.00	4,480	600	13.4	
	Fabric	150 SY	1.30	195	0	0.0	
	Rock Fence	150 LF	100.00	15,000	2,100	14.0	
02.1.2.C	Concrete						
	Shotcrete Slope Stabilization	300 SY	70.00	21,000	3,600	17.1	
	Retaining	60 CY	470.00	28,200	4,800	17.0	
	Minor Concrete Structures	8 CY	480.00	3,840	700	18.2	
02.1.3.-	Road Surfacing						
02.1.3.B	Site Work						
	Fine Grade Subgrade	26,000 SY	1.00	26,000	2,900	11.2	
	Sub Course	900 TON	11.00	9,900	1,100	11.1	
	Base Course	3,800 TON	16.00	60,800	6,700	11.0	
	Tack Coat	10 TON	300.00	3,000	300	10.0	
	Bituminous Surfacing	1,150 TON	43.00	49,450	5,400	10.9	
	Asphalt Dike, Type 5	5,000 LF	2.10	10,500	1,200	11.4	
02.1.R.-	Associated General Items:						
02.1.R.B	Site Work						
	Signs & Markings						
	Survey Monuments	35 EA	240.00	8,400	600	7.1	
	Roadside signs, Wood Posts	8 EA	210.00	1,680	100	6.0	
	Pavement Markings, Reflective	400 EA	4.20	1,680	100	6.0	
	Mile Post Markers	2 EA	60.00	120	0	0.0	
	Culvert Markers	50 EA	30.00	1,500	100	6.7	
	Striping	6,000 LF	0.30	1,800	100	5.6	
	Guard Rails						
	Metal Beam Guard Rail	1,860 LF	38.00	70,680	4,900	6.9	
	Cable Anchor Assembly	2 EA	925.00	1,850	100	5.4	
	Fencing & Gates						
	Gates, Vehicle	3 EA	940.00	2,820	200	7.1	
	Barb Wire Fence	3,000 LF	3.50	10,500	700	6.7	
	Cattle Guards	6 EA	4860.00	29,160	2,000	6.9	

	Landscaping & Irrigation							
	Seading, Hydraulic Slurry	1	ACRE	1880.00	1,880	100	5.3	
	Mulching, Hydraulic Slurry	1	JOB	LS	9,170	600	6.5	
	Plantings, Trees & Shrubs	350	EA	30.00	10,500	700	6.7	
	Maintenance of Landscaping	60	DAYS	98.00	5,880	400	6.8	
	Subtotal, Construction Costs:			$	2,778,185			
	Contingencies @ average of 15.2 % +/- *				$	421,815		(2.A)
02.1.-.-	**PONDEROSA ROAD**			**TOTAL:**	$	3,200,000		
	PONDEROSA BRIDGE							
02.1.A.-	Mob., Demob. and Preparatory Work:	1	JOB	LS	400,000	60,000	15.0	
02.1.J.-	**Bridges. Foundations:**							
02.1.J.B	Site Work							
	Clearing & Dental Slope Work	1	JOB	LS	11,000	1,900	17.3	
	Excavation, Common	5,000	CY	11.00	55,000	9,400	17.1	
	Excavation, Rock	4,000	CY	21.00	84,000	14,300	17.0	
	Excavation, Top Soil	500	CY	9.00	4,500	800	17.8	
	Excavation, Structure	4,300	CY	19.00	81,700	13,900	17.0	
	Structural Backfill	4,300	CY	17.00	73,100	12,400	17.0	
	Foundation Concrete	4,300	CY	230.00	989,000	227,500	23.0	(2.C)
	Reinforcing Steel	900,000	LBS	0.50	450,000	103,500	23.0	(2.C)
02.1.K.-	Bridge, Abutments and Piers:							
02.1.K.C	Concrete							
	Abutment #1	600	CY	330.00	198,000	45,500	23.0	(2.C)
	Abutment #2	600	CY	330.00	198,000	45,500	23.0	(2.C)
	Pier #1	3,600	CY	440.00	1,584,000	364,300	23.0	(2.C)
	Pier #2	3,600	CY	440.00	1,584,000	364,300	23.0	(2.C)
	Reinforcing Steel	2,400,000	LBS	0.50	1,200,000	276,000	23.0	(2.C)
02.1.L.-	Bridge, Superstructure and Deck:							
02.1.L.C	Concrete							
	Structural Concrete	6,400	CY	490.00	3,136,000	470,400	15.0	(2.A)
	Prestressing Steel	1,080,000	LBS	1.70	1,836,000	275,400	15.0	(2.A)
	Reinforcing Steel	1,600,000	LBS	0.50	800,000	120,000	15.0	(2.A)
02.1.L.E	Metals							
	Railings and Guards	40	TON	8,060	322,400	29,000	9.0	
	Bearings Pads	10	TON	5,430	54,300	4,900	9.0	
	Expansion Joint Assemblies	60	LF	770	46,200	4,200	9.1	
	Miscellaneous Steel	8,000	LBS	3.70	29,600	2,700	9.1	
02.1.L.L	Finishes							
	Painting	1	JOB	LS	10,000	900	9.0	
02.1.M.-	Bridge, Associated General Items:							
02.1.M.E	Metals							
	Bridge Barriers	1	JOB	LS	30,000	2,100	7.0	
	Signs	12	EA	160	1,920	100	5.2	
02.1.M.R	Electrical							
	Lighting	1	JOB	LS	200,000	14,000	7.0	
	Subtotal, Construction Costs:			$	13,378,720			
	Contingencies @ average of 18.1 % +/- *				$	2,421,280		(2.A)
02.1.-.-	**PONDEROSA BRIDGE**			**TOTAL:**	$	15,800,000		
02.3.-.-	**CEMETERIES, UTILITIES, AND STRUCTURES**							
	UTILITIES RELOCATIONS							
02.3.A.-	Mob., Demob. and Preparatory Work:	1	JOB	LS	50,000	5,500	11.0	

DETAILED ESTIMATE OF FIRST COST   TABLE 2   D6
Highway 49 / Ponderosa Road / Utilities Relocations

02.3.2.-	Utilities						
02.3.2.R	Electrical						
	Electrical						
	Transitition Power OH to UG	2 EA	9,700.00	19,400	2,500	12.9	
	Duct 4"	139,400 LF	7.40	1,031,560	134,100	13.0	
	Electrical Cable	142,200 LF	32.00	4,550,400	591,600	13.0	
	Concrete Encasement	520 CY	120.00	62,400	8,100	13.0	
	Manholes	70 EA	21,400.00	1,498,000	194,700	13.0	
	Cable Splices	210 EA	1,050.00	220,500	28,700	13.0	
	Telephone						
	Transitition Power OH to UG	2 EA	2,140.00	4,280	400	9.3	
	Duct 4"	69,700 LF	7.40	515,780	51,600	10.0	
	Electrical Cable	35,900 LF	32.00	1,148,800	114,900	10.0	
	Concrete Encasement	225 CY	120.00	27,000	2,700	10.0	
	Manholes	70 EA	9,740.00	681,800	68,200	10.0	
	Cable Splices	70 EA	260.00	18,200	1,800	9.9	
	Subtotal, Construction Costs:		$	9,828,120			
	Contingencies @ average of	11.9 % +/- *			$ 1,171,880		(2.A)
02.3.-.-	**UTILITIES RELOCATIONS**		**TOTAL:**		**$ 11,000,000**		
30.-.-.-	**PLANNING, ENGINEERING & DESIGN**	**HIGHWAY 49 & UTILITIES RELOCATION**					
30.D.-.-	ENVIRONMENTAL AND REGULATORY ACTIVITIES						
30.D.C.-	Supplemental EIS	220 DAYS	300.00	66,000			
30.D.2.-	401, 404, & ROD	30 DAYS	300.00	9,000			
30.D.Z.-	Contingencies	1 JOB LS			19,000		
30.E.-.-	DESIGN RELATED ENGINEERING						
30.E.1.-	Subsurface Explorations	1 JOB LS		215,000			
30.E.2.-	Sampling, Testing, & Analysis	1 JOB LS		500,000			
30.E.Z.-	Contingencies	1 JOB LS			178,800		
30.F.-.-	DESIGN DOCUMENT						
30.F.A.-	Draft Design Document	7,090 DAYS	310.00	2,197,900			
30.F.B.-	Final Design Document	1,770 DAYS	310.00	548,700			
30.F.F.-	Value Engineering (VE) Studies	200 DAYS	310.00	62,000			
30.F.Z.-	Contingencies	1 JOB LS			702,400		
30.G.-.-	FEATURE DESIGN MEMORANDUM (MITIGATION)						
30.G.A.-	Draft FDM	250 DAYS	310.00	77,500			
30.G.B.-	Final FDM	60 DAYS	310.00	18,600			
30.G.Z.-	Contingencies	1 JOB LS			24,600		
30.H.-.-	PLANS AND SPECIFICATIONS						
30.H.A.-	Preliminary Design	3,470 DAYS	310.00	1,075,700			
30.H.B.-	Final Design	920 DAYS	310.00	285,200			
30.H.C.-	Design Revisions	230 DAYS	310.00	71,300			
30.H.E.-	BCO Review	150 DAYS	310.00	46,500			
30.H.Z.-	Contingencies	1 JOB LS			369,300		
30.J.-.-	ENGINEERING DURING CONSTRUCTION						
30.J.H.-	Value Engineering Change Proposals	50 DAYS	310.00	15,500			
30.J.1.-	Review of E&D by Constr. Contractor	20 DAYS	310.00	6,200			
30.J.2.-	Periodic Inspections	70 DAYS	310.00	21,700			
30.J.9.-	All Other Engr. During Construction	100 DAYS	310.00	31,000			
30.J.Z.-	Contingencies	1 JOB LS			18,800		
30.M.-.-	COST ENGINEERING	860 DAYS	310.00	266,600			
30.P.-.-	PROJECT MANAGEMENT	2,600 DAYS	400.00	1,040,000			
30.P.Z.-	Contingencies	1 JOB LS			260,400		

DETAILED ESTIMATE OF FIRST COST TABLE 2 D7
Highway 49 / Ponderosa Road / Utilities Relocations

30.Z.-.-	MISCELLANEOUS ACTIVITIES					
30.Z.1.-	FWS Support	1 JOB LS		87,500		
30.Z.1.-	Surveys (Topographical)	1 JOB LS		95,000		
30.Z.1.-	Surveys (Cultural)	1 JOB LS		50,000		
30.Z.Z.-	Contingencies:	1 JOB LS			58,100	
	Subtotal, Construction Costs:			$ 6,786,900		
	Contingencies @ average of 23.8 % +/- *				$ 1,613,100	(30.B)
30.-.-.-	**PLANNING, ENGINEERING & DESIGN**		**TOTAL:**		**$ 8,400,000**	
30.-.-.-	**PLANNING, ENGINEERING & DESIGN**	**PONDEROSA WAY BRIDGE RELOCATION**				
30.D.-.-	ENVIRONMENTAL AND REGULATORY ACTIVITIES					
30.D.C.-	Supplemental EIS	132 DAYS	300.00	39,600		
30.D.Z.-	Contingencies	1 JOB LS			9,900	
30.E.-.-	DESIGN RELATED ENGINEERING					
30.E.1.-	Subsurface Explorations	1 JOB LS		79,000		
30.E.2.-	Sampling, Testing, & Analysis	1 JOB LS		54,500		
30.E.Z.-	Contingencies	1 JOB LS			33,400	
30.F.-.-	DESIGN DOCUMENT					
30.F.A.-	Draft Design Document	1,810 DAYS	310.00	561,100		
30.F.B.-	Final Design Document	450 DAYS	310.00	139,500		
30.F.F.-	Value Engineering (VE) Studies	100 DAYS	310.00	31,000		
30.F.Z.-	Contingencies	1 JOB LS			183,500	
30.G.-.-	FEATURE DESIGN MEMORANDUM (MITIGATION)					
30.G.A.-	Draft FDM	30 DAYS	310.00	9,300		
30.G.B.-	Final FDM	10 DAYS	310.00	3,100		
30.G.Z.-	Contingencies	1 JOB LS			3,000	
30.H.-.-	PLANS AND SPECIFICATIONS					
30.H.A.-	Preliminary Design	960 DAYS	310.00	297,600		
30.H.B.-	Final Design	250 DAYS	310.00	77,500		
30.H.C.-	Design Revisions	60 DAYS	310.00	18,600		
30.H.E.-	BCO Review	50 DAYS	310.00	15,500		
30.H.Z.-	Contingencies	1 JOB LS			102,500	
30.J.-.-	ENGINEERING DURING CONSTRUCTION					
30.J.2.-	Periodic Inspections	70 DAYS	310.00	21,700		
30.J.9.-	All Other Engr. During Construction	60 DAYS	310.00	18,600		
30.J.Z.-	Contingencies	1 JOB LS			10,100	
30.M.-.-	COST ENGINEERING	350 DAYS	310.00	108,500		
30.P.-.-	PROJECT MANAGEMENT	1,060 DAYS	400.00	424,000		
30.P.Z.-	Contingencies	1 JOB LS			105,600	
30.Z.-.-	MISCELLANEOUS ACTIVITIES					
30.Z.1.-	FWS Support	1 JOB LS		50,500		
30.Z.1.-	Surveys (Topographical)	1 JOB LS		49,000		
30.Z.1.-	Surveys (Cultural)	1 JOB LS		30,000		
30.Z.Z.-	Contingencies:	1 JOB LS			32,400	
	Subtotal, Construction Costs:			$ 2,028,600		
	Contingencies @ average of 23.2 % +/- *				$ 471,400	(30.B)
30.-.-.-	**PLANNING, ENGINEERING & DESIGN**		**TOTAL:**		**$ 2,500,000**	
31.-.-.-	**CONSTRUCTION MANAGEMENT (S & I)**					
31.B.-.-	CONTRACT ADMINISTRATION					
31.B.1.-	Pre-award Activities					
31.B.1.1	Resident Office	718 MH	45.00	32,310	3,600	
31.B.1.2	District Office	1,787 MH	60.00	107,220	12,000	

31.B.2.-	Award Activities	447 MH	60.00	26,820	3,000	
31.B.3.-	Review/Approval of Contract Payment	4,308 MH	45.00	193,860	22,000	
31.B.4.-	Contract Modifications					
31.B.4.1	Resident Office	28,723 MH	45.00	1,292,535	143,000	
31.B.4.2	District Office	2,234 MH	60.00	134,040	15,000	
31.B.5.-	Progress and Completion Reports	2,872 MH	45.00	129,240	14,000	
31.D.-.-	REVIEW OF SHOP DRAWINGS					
31.D.1	Resident Office	14,362 MH	45.00	646,290	72,000	
31.D.2	District Office	2,234 MH	60.00	134,040	15,000	
31.E.-.-	INSPECTION AND QUALITY ASSURANCE					
31.E.1.-	Schedule Compliance	2,872 MH	45.00	129,240	14,000	
31.E.2.-	Compliance Sampling & Testing					
31.E.2.1	Resident Office	10,771 MH	45.00	484,695	54,000	
31.E.2.2	Laboratory Charges	1 JOB LS		484,704	54,000	
31.E.9.-	Quality Assurance Personnel	36,622 MH	45.00	1,647,990	183,000	
31.F.-.-	PROJECT OFFICE OPERATION					
31.F.1.-	Resident Office	2,872 MH	45.00	129,240	14,000	
31.F.2.-	Vehicles and Equipment	1 JOB LS		1,293,324	144,000	
31.H.-.-	CONTRACTOR CLAIMS & LITIGATIONS	7,818 MH	60.00	469,080	52,000	
31.P.-.-	PROJECT MANAGEMENT	7,818 MH	60.00	469,080	52,000	
	Subtotal, Construction Costs:		$	7,803,708		
	Contingencies @ average of 11.1 % +/- *			$	866,292	(31.A)
31.-.-.-	**CONSTRUCTION MANAGEMENT (S & I)**		**TOTAL:**	**$**	**8,670,000**	
02.A.-.-	**ROADS, Real Estate Activities**					
02.A.D.-	Attorneys' Opinion of Compensability					
02.A.D.G	Relocation Contract	10 MD	450.00	4,500	700	
02.A.D.Z	All Other	5 MD	460.00	2,300	300	
02.A.E.-	Mapping, Surveying and Tract Ownership					
02.A.E.D	Prepare Documents	2 MM	8,300.00	16,600	2,500	
02.A.E.E	Review of Documents	1 MM	8,300.00	8,300	1,200	
02.A.E.F	Review for Compliance	1 MM	8,300.00	8,300	1,200	
02.A.F.-	Title Evidence					
02.A.F.D	Prepare Documents	1 MM	8,300.00	8,300	1,200	
02.A.F.E	Review of Documents	5 MD	460.00	2,300	300	
02.A.G.-	Negotiations and Closings					
02.A.G.D	Prepare Documents	1 MM	8,300.00	8,300	1,200	
02.A.G.E	Review of Documents	5 MD	460.00	2,300	300	
02.A.G.F	Review for Compliance	1 MM	8,300.00	8,300	1,200	
02.A.H.-	Condemnation (Pre-DT Filing)					
02.A.H.D	Prepare Documents	5 MD	460.00	2,300	300	
02.A.H.E	Review of Documents	3 MD	460.00	1,380	200	
02.A.H.F	Review for Compliance	1 MD	460.00	460	100	
02.A.J.-	Condemnation (Post-DT Filing)					
02.A.J.D	Prepare Documents	10 MD	450.00	4,500	700	
02.A.J.E	Review of Documents	5 MD	460.00	2,300	300	
02.A.J.F	Review for Compliance	1 MD	500.00	500	100	

DETAILED ESTIMATE OF FIRST COST TABLE 2 D9

Highway 49 / Ponderosa Road / Utilities Relocations

02.A.K.-	Staff Appraisals						
02.A.K.H	Prepare Appraisals	1 MM	8,300.00	8,300	1,200		
02.A.K.J	Review Appraisals	5 MD	460.00	2,300	300		
02.A.T.-	Land Payments	1 LS		576,500	130,000		
02.A.V.-	Claim Payments	1 LS		58,000	10,000		
	Subtotal, Construction Costs:		$	726,040			
	Contingencies @ average of 21.2 % +/- *			$	153,960		(1.A)
02.A.-.-	**ROADS, Real Estate Activities**		**TOTAL:**	**$**	**880,000**		
02.C.-.-	**CEMETERIES, UTILITIES & STRUCTURES, Real Estate Act.**						
02.C.D.-	Attorneys' Opinion of Compensability						
02.C.D.G	Relocation Contract	5 MD	400.00	2,000	300	15.0	
02.C.D.Z	All Other	5 MD	400.00	2,000	300	15.0	
02.C.E.-	Mapping, Surveying and Tract Ownership						
02.C.E.D	Prepare Documents	12 MD	420.00	5,040	800	15.9	
02.C.E.E	Review of Documents	12 MD	420.00	5,040	800	15.9	
02.C.E.F	Review for Compliance	5 MD	400.00	2,000	300	15.0	
02.C.E.Z	All Other	5 MD	400.00	2,000	300	15.0	
02.C.F.-	Title Evidence						
02.C.F.D	Prepare Documents	12 MD	420.00	5,040	800	15.9	
02.C.F.E	Review of Documents	5 MD	400.00	2,000	300	15.0	
02.C.F.F	Review for Compliance	5 MD	400.00	2,000	300	15.0	
02.C.F.Z	All Other	5 MD	400.00	2,000	300	15.0	
02.C.G.-	Negotiations and Closings						
02.C.G.D	Prepare Documents	72 MD	420.00	30,240	4,500	14.9	
02.C.G.E	Review of Documents	24 MD	420.00	10,080	1,500	14.9	
02.C.G.F	Review for Compliance	5 MD	400.00	2,000	300	15.0	
02.C.G.Z	All Other	5 MD	400.00	2,000	300	15.0	
02.C.H.-	Condemnation (Pre-DT Filing)						
02.C.H.D	Prepare Documents	5 MD	400.00	2,000	300	15.0	
02.C.H.E	Review of Documents	5 MD	400.00	2,000	300	15.0	
02.C.H.F	Review for Compliance	5 MD	400.00	2,000	300	15.0	
02.C.H.Z	All Other	5 MD	400.00	2,000	300	15.0	
02.C.J.-	Condemnation (Post-DT Filing)						
02.C.J.D	Prepare Documents	5 MD	400.00	2,000	300	15.0	
02.C.J.E	Review of Documents	5 MD	400.00	2,000	300	15.0	
02.C.J.F	Review for Compliance	5 MD	400.00	2,000	300	15.0	
02.C.J.Z	All Other	5 MD	400.00	2,000	300	15.0	
02.C.L.-	Contract Appraisals						
02.C.L.H	Prepare Appraisals	5 MD	400.00	2,000	300	15.0	
02.C.L.J	Review Appraisals	5 MD	400.00	2,000	300	15.0	
02.C.L.Z	All Other	5 MD	400.00	2,000	300	15.0	
	Subtotal, Construction Costs:		$	95,440			
	Contingencies @ average of 15.3 % +/- *			$	14,560		(1.A)
02.C.-.-	**CEMETERIES, UTILITIES & STRUCTURES, Real Estate Act.**		**TOTAL:**	**$**	**110,000**		

FOUNDATION AND ACCESS ROADS CONTRACT

COST ESTIMATE
DETAILED ESTIMATE OF FIRST COST TABLE 2 B1

ACCOUNT NUMBER	ITEM	QUANTITY	UNIT	UNIT PRICE $	AMOUNT $	CONTINGENCY $ *	CONTINGENCY % *	REASON
	Effective Price Date(EPD) 1-Oct-91					* (Figures Rounded)		
	Dam Foundation and Access Roads							
	FEDERAL							
01.-.-.-	**LANDS AND DAMAGES**							
01.B.-.-	POST-AUTHORIZATION PLANNING							
01.B.1.-	Develop Cost Estimate	126	MD	440.00	55,440			
01.B.3.-	Real estate design Memorandum	28	MD	450.00	12,600			
01.B.4.-	Evaluate Sponsor Capability	4	MD	460.00	1,840			
01.B.8.-	All Other	228	MD	430.00	98,040			
01.B.9.-	Contingencies					25,200		
01.C.-.-	LOCAL COOPERATION AGREEMENT							
01.C.2.-	Final LCA	6	MD	500.00	3,000			
01.C.9.-	Contingencies					450		
01.D.-.-	ACQUISITIONS							
01.D.2.-	Mapping, Survey and Tract Ownership							
01.D.2.D	Prepare Documents	21	MD	350.00	7,350			
01.D.2.Z	All Other	25	MD	350.00	8,750			
01.D.4.-	Negotiations and Closing							
01.D.4.D	Prepare Documents	91	MD	470.00	42,770			
01.D.4.F	Review for Compliance	38	MD	430.00	16,340			
01.D.4.Z	All Other	102	MD	460.00	46,920			
01.D.9.-	Contingencies					18,400		
01.F.-.-	APPRAISALS							
01.F.1.-	Staff Appraisals							
01.F.1.Z	All Other	3	MD	500.00	1,500			
01.F.2.-	Contract Appraisals							
01.F.2.Z	All Other	77	MD	500.00	38,500			
01.F.9.-	Contingencies					6,000		
	Subtotal, Construction Costs:			$	333,050			
	Contingencies @ average of	14.1 % +/- *			$	46,950		(1.A)
01.-.-.-	**LANDS AND DAMAGES**			**TOTAL:**	$	**380,000**		
04.-.-.-	**DAMS**							
04.1.-.-	MAIN DAM - RCC							
04.1.A.-	Mob., Demob. and Preparatory Work:	1	LS	1,100,000	1,100,000	280,000	25.5	(4.A)
04.1.D.-	Earthwork for Structures:							
04.1.D.B	Site Work							
	Clearing and Grubbing	15	Acre	2,000.00	30,000	10,000	33.3	(4.A)
	Excavation, Unclassified	1,287,000	CY	4.00	5,148,000	1,030,000	20.0	(4.B)
	Excavation, Rock	530,000	CY	8.00	4,240,000	1,060,000	25.0	(4.B)
	Excavation, Streambed	300,000	CY	4.00	1,200,000	240,000	20.0	(4.B)
04.1.E.-	Foundation Work:							
04.1.E.B	Site Work							
	Foundation Treatment							
	Excavation of "J" Block Area	304,000	CY	20.00	6,080,000	1,820,000	29.9	(4.B,C)
	Excavation of Fault, Talc, and Shear Zones	30,400	CY	50.00	1,520,000	610,000	40.1	(4.B,C)
	Excavation of Slide Area	1,630,000	CY	6.00	9,780,000	2,450,000	25.1	(4.C)
	Cleanup open cut foundation	170,000	SY	5.00	850,000	210,000	24.7	(4.A)
	Cleanup Fault-Talc-Shear Zone	31,000	SY	8.00	248,000	60,000	24.2	(4.A)
	Consolidation Grouting							
	Drill Holes	23,000	LF	12.00	276,000	70,000	25.4	(4.C)
	Additional Drilling (Nipples)	3,450	LF	6.00	20,700	5,000	24.2	(4.C)
	Hookups for Grout Pipes	1,150	Each	50.00	57,500	14,000	24.3	(4.C)

DETAILED ESTIMATE OF FIRST COST TABLE 2 B2

Dam Foundation and Access Roads

	Setups for Drilling	1,150 Each	140.00	161,000	40,000	24.8	(4.C)
	Cement	6,900 CF	5.40	37,260	20,000	53.7	(4.C)
	Pressure Tests	575 Hrs	140.00	80,500	20,000	24.8	(4.C)
04.1.E.C	Concrete (Dental)						
	Concrete in "J" Block Area	304,000 CY	80.00	24,320,000	6,080,000	25.0	(4.B,C)
	Concrete in Fault, Talc, and Shear Zones	30,400 CY	80.00	2,432,000	1,000,000	41.1	(4.B,C)
	Subtotal, Construction Costs:			$ 57,580,960			
	Contingency rounded to an average of	26.1 % +/- *			$ 15,019,040		
04.1.-.-	**MAIN DAM - RCC**		**SUBTOTAL:**		**$ 72,600,000**		
04.3.-.-	**OUTLET WORKS**						
	MODIFIED DIVERSION TUNNEL						
04.3.B.-	Care and Diversion of Water:						
04.3.B.B	Site Work						
	Cofferdam						
	Channel Excavation, Common	500,000 CY	4.00	2,000,000	300,000	15.0	(4.A,B)
	Earthwork Embankment	40,000 CY	4.50	180,000	27,000	15.0	(4.A,B)
	Cofferdam Removal	40,000 CY	4.00	160,000	24,000	15.0	(4.A,B)
04.3.D.-	Earthwork for Structures:						
04.3.D.B	Site Work						
	Excavation, Unclassified	600 CY	9.00	5,400	1,350	25.0	(4.A)
04.3.4.-	Intake Structure:						
04.3.4.C	Concrete						
	Concrete in Place						
	Walls and Slabs	2,700 CY	290.00	783,000	234,900	30.0	(4.A)
	Trashracks	30 CY	600.00	18,000	5,400	30.0	(4.A)
	Cement	17,200 Cwt	4.00	68,800	20,640	30.0	(4.A)
	Reinforcing Steel	276,000 Lbs	0.50	138,000	41,400	30.0	(4.A)
04.3.5.-	Intake Gates and Equipment:						
04.3.5.E	Metals						
	High Head Bulkhead gates 10'Wx15'H incl. frames and guides	2 Each	400,000.00	800,000	160,000	20.0	(04.A)
	Subtotal, Construction Costs:			$ 4,153,200			
	Contingency rounded to an average of	20.4 % +/- *			$ 846,800		
04.3.-.-	**OUTLET WORKS**		**SUBTOTAL:**		**$ 5,000,000**		
08.-.-.-	**ROADS, RAILROADS, AND BRIDGES**						
08.2.-.-	**ROADS (Including Bridges)**						
	ACCESS ROAD TO TOP OF DAM						
08.2.2.-	Construct Roadbed To Sugrade:						
08.2.2.B	Site Work						
	Excavation, Common	36,500 CY	7.00	255,500	38,325	15.0	(4.A)
	Dust and Erosion Control	7 Acre	1,100.00	7,700	1,155	15.0	(4.A)
08.2.G.-	Drainage:						
08.2.G.B	Site Work						
	Drainage	1 LS	20,000.00	20,000	5,000	25.0	(4.A)
08.2.3.-	Road Surfacing:						
08.2.3.B	Site Work						
	Base Course	3,500 Ton	16.00	56,000	8,400	15.0	(4.A)
	Bituminous Surface Course	1,900 Ton	46.00	87,400	13,110	15.0	(4.A)
	Tack Coat	5 Ton	260.00	1,300	195	15.0	(4.A)
	Bituminous Dike	5,600 LF	3.00	16,800	4,200	25.0	(4.A)

08.2.R.-	Associated General Items:						
08.2.R.B	Site Work						
	Guard Rails	2,100 LF	23.00	48,300	12,075	25.0	(4.A)
	ACCESS ROAD TO INLET GATES						
08.2.2.-	Construct Roadbed To Sugrade:						
08.2.2.B	Site Work						
	Excavation, Common	85,000 CY	7.00	595,000	89,250	15.0	(4.A)
	Dust and Erosion Control	4 Acre	1,100.00	4,400	660	15.0	(4.A)
08.2.G.-	Drainage:						
08.2.G.B	Site Work						
	Drainage	1 LS	30,000.00	30,000	7,500	25.0	(4.A)
08.2.3.-	Road Surfacing:						
08.2.3.B	Site Work						
	Base Course	3,050 Ton	16.00	48,800	7,320	15.0	(4.A)
	ACCESS ROAD TO OUTLET WORKS						
08.2.2.-	Construct Roadbed To Sugrade:						
08.2.2.B	Site Work						
	Excavation, Common	48,000 CY	7.00	336,000	50,400	15.0	(4.A)
	Dust and Erosion Control	8 Acre	1,100.00	8,800	1,320	15.0	(4.A)
08.2.G.-	Drainage:						
08.2.G.B	Site Work						
	Drainage	1 LS	30,000.00	30,000	7,500	25.0	(4.A)
08.2.3.-	Road Surfacing:						
08.2.3.B	Site Work						
	Base Course	4,000 Ton	16.00	64,000	9,600	15.0	(4.A)
	Bituminous Surface Course	2,000 Ton	46.00	92,000	13,800	15.0	(4.A)
	Tack Coat	5 Ton	260.00	1,300	195	15.0	(4.A)
	Bituminous Dike	6,700 LF	3.00	20,100	5,025	25.0	(4.A)
08.2.R.-	Associated General Items:						
08.2.R.B	Site Work						
	Guard Rails	1,700 LF	23.00	39,100	9,775	25.0	(4.A)
	Subtotal, Construction Costs:		$	1,762,500			
	Contingency rounded to an average of	13.5 % +/- *		$	237,500		
08.-.-.-	**ROADS, RAILROADS, AND BRIDGES**		**TOTAL:**	$	**2,000,000**		
30.-.-.-	**PLANNING, ENGINEERING & DESIGN**						
30.D.-.-	ENVIRONMENTAL AND REGULATORY ACTIVITIES						
30.D.C.-	Supplemental EIS	440 DAYS	300.00	132,000			
30.D.Z.-	Contingencies	1 JOB LS			33,100		
30.E.-.-	DESIGN RELATED ENGINEERING						
30.E.1.-	Subsurface Explorations	1 JOB LS		785,000			
30.E.2.-	Sampling, Testing, & Analysis	1 JOB LS		395,000			
30.E.Z.-	Contingencies	1 JOB LS			295,000		
30.F.-.-	LETTER REPORT						
30.F.A.-	Draft Letter Report	330 DAYS	310	102,300			
30.F.B.-	Final Letter Report	80 DAYS	310	24,800			
30.F.Z.-	Contingencies	1 JOB LS			32,400		
30.G.-.-	FEATURE DESIGN MEMORANDUM - ACCESS ROADS						
30.G.A.-	Draft Feature Design Memorandum	1330 DAYS	310	412,300			
30.G.B.-	Final FDM	330 DAYS	310	102,300			
30.G.F.-	Value Engineering	100 DAYS	310	31,000			
30.G.Z.-	Contingencies	1 JOB LS			129,200		
30.G.-.-	FEATURE DESIGN MEMORANDUM - SITE GEOLOGY						
30.G.A.-	Draft Feature Design Memorandum	4460 DAYS	310	1,382,600			
30.G.B.-	Final FDM	1120 DAYS	310	347,200			
30.G.Z.-	Contingencies	1 JOB LS			432,500		
30.G.-.-	FEATURE DESIGN MEMORANDUM - DIVERSION WORKS						
30.G.A.-	Draft Feature Design Memorandum	850 DAYS	310	263,500			
30.G.B.-	Final FDM	210 DAYS	310	65,100			
30.G.F.-	Value Engineering	130 DAYS	310	40,300			

30.G.Z.-	Contingencies	1 JOB LS			82,100	
30.H.-.-	PLANS AND SPECIFICATIONS					
30.H.A.-	Preliminary Design	1540 DAYS	310	477,400		
30.H.B.-	Final Design	410 DAYS	310	127,100		
30.H.C.-	Design Revisions	100 DAYS	310	31,000		
30.H.E.-	BCO Review	60 DAYS	310	18,600		
30.H.Z.-	Contingencies	1 JOB LS			163,900	
30.J.-.-	ENGINEERING DURING CONSTRUCTION					
30.J.2.-	Periodic Inspections	240 DAYS	310	74,400		
30.J.9.-	All Engineering During Constr	190 DAYS	310	58,900		
30.J.Z.-	Contingencies	1 JOB LS			33,800	
30.M.-.-	COST ENGINEERING	610 DAYS	310	189,100		
30.P.-.-	PROJECT MANAGEMENT	1710 DAYS	400	684,000		
30.P.Z.-	Contingencies	1 JOB LS			171,400	
30.Z.-.-	MISCELLANEOUS ACTIVITIES					
30.Z.1.-	FWS Support	1 JOB LS		120,000		
30.Z.1.-	Surveys (Topographical)	1 JOB LS		170,000		
30.Z.1.-	Surveys (Cultural)	1 JOB LS		100,000		
30.Z.Z.-	Contingencies:	1 JOB LS			97,500	
	Subtotal, Construction Costs:		$	6,133,900		
	Contingency rounded to an average of	23.9 % +/- *		$	1,466,100	(30.B)
30.-.-.-	**PLANNING, ENGINEERING & DESIGN**		**Subtotal:**	**$**	**7,600,000**	
31.-.-.-	**CONSTRUCTION MANAGEMENT (S & I)**					
31.B.-.-	CONTRACT ADMINISTRATION					
31.B.1.-	Pre-award Activities					
31.B.1.1	Resident Office	587 MH	45.00	26,415	3,000	
31.B.1.2	District Office	1,461 MH	60.00	87,660	10,000	
31.B.2.-	Award Activities	365 MH	60.00	21,900	2,000	
31.B.3.-	Review/Approval of Contract Payment	3,523 MH	45.00	158,535	18,000	
31.B.4.-	Contract Modifications					
31.B.4.1	Resident Office	23,485 MH	45.00	1,056,825	117,000	
31.B.4.2	District Office	1,826 MH	60.00	109,560	12,000	
31.B.5.-	Progress and Completion Reports	2,349 MH	45.00	105,705	12,000	
31.D.-.-	REVIEW OF SHOP DRAWINGS					
31.D.1	Resident Office	11,743 MH	45.00	528,435	59,000	
31.D.2	District Office	1,826 MH	60.00	109,560	12,000	
31.E.-.-	INSPECTION AND QUALITY ASSURANCE					
31.E.1.-	Schedule Compliance	2,349 MH	45.00	105,705	12,000	
31.E.2.-	Compliance Sampling & Testing					
31.E.2.1	Resident Office	8,807 MH	45.00	396,315	44,000	
31.E.2.2	Laboratory Charges	1 JOB LS		396,317	44,000	
31.E.9.-	Quality Assurance Personnel	29,944 MH	45.00	1,347,480	150,000	
31.F.-.-	PROJECT OFFICE OPERATION					
31.F.1.-	Resident Office	2,349 MH	45.00	105,705	12,000	
31.F.2.-	Vehicles and Equipment	1 JOB LS		1,057,483	117,000	
31.H.-.-	CONTRACTOR CLAIMS & LITIGATIONS	6,393 MH	60.00	383,580	43,000	
31.P.-.-	PROJECT MANAGEMENT	6,393 MH	60.00	383,580	43,000	
	Subtotal, Construction Costs:		$	6,380,760		
	Contingency rounded to an average of	11.3 % +/- *		$	719,240	(31.A)
31.-.-.-	**CONSTRUCTION MANAGEMENT (S & I)**		**TOTAL:**	**$**	**7,100,000**	

DETAILED ESTIMATE OF FIRST COST TABLE 2 B5

Dam Foundation and Access Roads

NON-FEDERAL

01.-.-.-	**LANDS AND DAMAGES**					
01.B.-.-	POST-AUTHORIZATION PLANNING					
01.B.1.-	Develop Cost Estimate	32 MD	450.00	14,400		
01.B.2.-	Develop Acquisiton Schedule	29 MD	450.00	13,050		
01.B.9.-	Contingencies				2,800	
01.D.-.-	ACQUISITIONS					
01.D.1.-	Attorney's Opinion					
01.D.1.E	Review of Documents	17 MD	450.00	7,650		
01.D.1.Z	All Other	32 MD	450.00	14,400		
01.D.2.-	Mapping, Survey and Tract Ownership					
01.D.2.D	Prepare Documents	249 MD	450.00	112,050		
01.D.2.E	Review of Documents	188 MD	450.00	84,600		
01.D.2.F	Review for Compliance	94 MD	450.00	42,300		
01.D.2.Z	All Other	94 MD	450.00	42,300		
01.D.3.-	Title Evidence					
01.D.3.D	Prepare Documents	84 MD	450.00	37,800		
01.D.3.Z	All Other	41 MD	450.00	18,450		
01.D.4.-	Negotiations and Closing					
01.D.4.D	Prepare Documents	484 MD	450.00	217,800		
01.D.4.E	Review of Documents	197 MD	450.00	88,650		
01.D.4.F	Review for Compliance	98 MD	450.00	44,100		
01.D.4.Z	All Other	116 MD	450.00	52,200		
01.D.5.-	Condemnation (Pre-DT Filing)					
01.D.5.D	Prepare Documents	42 MD	450.00	18,900		
01.D.5.E	Review of Documents	17 MD	450.00	7,650		
01.D.5.F	Review for Compliance	8 MD	450.00	3,600		
01.D.5.Z	All Other	8 MD	450.00	3,600		
01.D.9.-	Contingencies				79,600	
01.E.-.-	CONDEMNATION (POST-DT FILING)					
01.E.0.D	Prepare Documents	34 MD	450.00	15,300		
01.E.0.E	Review of Documents	8 MD	450.00	3,600		
01.E.0.F	Review for Compliance	8 MD	450.00	3,600		
01.E.0.Z	All Other	1 LS		67,200		
01.E.9.-	Contingencies				9,000	
01.F.-.-	APPRAISALS					
01.F.1.-	Staff Appraisals					
01.F.1.H	Prepare Documents	312 MD	450.00	140,400		
01.F.1.J	Review of Documents	125 MD	450.00	56,250		
01.F.1.F	Review for Compliance	125 MD	450.00	56,250		
01.F.1.Z	All Other	63 MD	450.00	28,350		
01.F.2.-	Contract Appraisals					
01.F.2.H	Prepare Documents	1 LS		16,800		
01.F.2.J	Review of Documents	38 MD	450.00	17,100		
01.F.9.-	Contingencies				31,500	
01.K.-.-	TEMPORARY PERMITS					
01.K.0.D	Prepare Documents	34 MD	450.00	15,300		
01.K.0.Z	All Other	17 MD	450.00	7,650		
01.K.9.-	Contingencies				2,300	
01.M.-.-	REAL ESTATE RECEIPTS/PAYMENTS					
01.M.3.-	Land Payments	1 LS		33,247,000		
01.M.5.-	Damage Payments	1 LS		3,325,000		
01.M.9.-	Contingencies				8,312,000	
	Subtotal, Construction Costs:			$ 37,823,300		
	Contingencies @ average of 22.4 % +/- *			$	8,476,700	(1.B)
01.-.-.-	**LANDS AND DAMAGES**		**TOTAL:**	$	**46,300,000**	

MAIN DAM CONTRACT

COST ESTIMATE

DETAILED ESTIMATE OF FIRST COST TABLE 2 C1

ACCOUNT NUMBER	ITEM	QUANTITY	UNIT	UNIT PRICE $	AMOUNT $	CONTINGENCY $ *	CONTINGENCY % *	REASON
	Effective Price Date(EPD) 1-Oct-91 Main Dam					* (Figures Rounded)		
	FEDERAL							
01.-.-.-	**LANDS AND DAMAGES**							
01.B.-.-	POST-AUTHORIZATION PLANNING							
01.B.1.-	Develop Cost Estimate	90	MD	440.00	39,600			
01.B.3.-	Real estate design Memorandum	20	MD	450.00	9,000			
01.B.4.-	Evaluate Sponsor Capability	3	MD	530.00	1,590			
01.B.8.-	All Other	163	MD	430.00	70,090			
01.B.9.-	Contingencies					18,200		
01.C.-.-	LOCAL COOPERATION AGREEMENT							
01.C.2.-	Final LCA	4	MD	500.00	2,000			
01.C.9.-	Contingencies					300		
01.D.-.-	ACQUISITIONS							
01.D.1.-	Attorney's Opinion							
01.D.1.F	Review for Compliance	7	MD	530.00	3,710			
01.D.2.-	Mapping, Survey and Tract Ownership							
01.D.2.D	Prepare Documents	10	MD	350.00	3,500			
01.D.2.F	Review for Compliance	5	MD	360.00	1,800			
01.D.2.Z	All Other	18	MD	350.00	6,300			
01.D.4.-	Negotiations and Closing							
01.D.4.D	Prepare Documents	60	MD	470.00	28,200			
01.D.4.F	Review for Compliance	30	MD	420.00	12,600			
01.D.4.Z	All Other	73	MD	460.00	33,580			
01.D.9.-	Contingencies					13,400		
01.F.-.-	APPRAISALS							
01.F.1.-	Staff Appraisals							
01.F.1.Z	All Other	2	MD	500.00	1,000			
01.F.2.-	Contract Appraisals							
01.F.2.Z	All Other	55	MD	500.00	27,500			
01.F.9.-	Contingencies					4,300		
	Subtotal, Construction Costs:			$	240,470			
	Contingencies @ average of	16.4 % +/- *			$	39,530		(1.A)
01.-.-.-	**LANDS AND DAMAGES**			**TOTAL:**	$	280,000		
03.-.-.-	**RESERVOIRS**							
03.0.2.-	Boundary Line Survey and Marking:							
03.0.2.A	General Requirements							
	Surveying	1	LS	60,000.00	60,000	15,000	25.0	(3.A)
	Boundary Fence	105,600	LF	3.00	316,800	79,200	25.0	(3.A)
	Subtotal, Construction Costs:			$	376,800			
	Contingency rounded to an average of	24.7 % +/- *			$	93,200		
03.-.-.-	**RESERVOIRS**			**TOTAL:**	$	470,000		

DETAILED ESTIMATE OF FIRST COST TABLE 2 C2

Main Dam

04.-.-.-	DAMS							
04.1.-.-	**MAIN DAM (RCC)**							
04.1.A.-	Mob., Demob. and Preparatory Work:	1	LS	2,600,000	2,600,000	390,000	15.0	(4.A)
04.1.F.-	Seepage Control:							
04.1.F.B	Site Work							
	Drilling and Grouting							
	Drilling Grout Holes							
	Primary Holes	46,800	LF	12.00	561,600	56,160	10.0	(4.A)
	Secondary Holes	23,200	LF	12.00	278,400	69,600	25.0	(4.A)
	Hookups for Grout Pipes	1,400	Each	50.00	70,000	10,500	15.0	(4.A)
	Setups for Drilling	1,400	Each	140.00	196,000	29,400	15.0	(4.A)
	Re-drill	35,000	LF	9.00	315,000	78,750	25.0	(4.A)
	Cement	19,500	CF	5.40	105,300	26,325	25.0	(4.A)
	Pressure Tests	1,400	Hrs	140.00	196,000	49,000	25.0	(4.A)
04.1.G.-	Drainage							
04.1.G.B	Site Work							
	Drain Holes	35,100	LF	18.00	631,800	157,950	25.0	(4.A)
04.1.1.-	Concrete Dam, Non Overflow Section:							
04.1.1.C	Concrete (RCC)							
	Concrete	2,530,000	CY	18.00	45,540,000	9,108,000	20.0	(4.E)
	Cement	4,554,000	Cwt	4.00	18,216,000	3,643,200	20.0	(4.E)
	Pozzolan	3,390,000	Cwt	2.60	8,814,000	1,762,800	20.0	(4.E)
	Contraction Joint Steel (Dam)	596,000	SF	5.00	2,980,000	596,000	20.0	(4.A)
	Waterstops (Dam)	4,400	LF	8.00	35,200	7,040	20.0	(4.A)
	Pre-cast Concrete							
	Grout Gallery	5,300	CY	290.00	1,537,000	230,550	15.0	(4.A)
	Access Gallery	3,425	CY	290.00	993,250	148,988	15.0	(4.A)
	Cement	54,750	Cwt	4.00	219,000	32,850	15.0	(4.A)
	Reinforcing Steel	1,308,800	Lbs	0.50	654,400	98,160	15.0	(4.A)
	Backfill Concrete	14,450	CY	80.00	1,156,000	173,400	15.0	(4.A)
	Cement	90,600	Cwt	4.00	362,400	54,360	15.0	(4.A)
	Conventional Concrete							
	Concrete Slab (Top of Dam)	3,700	CY	90.00	333,000	49,950	15.0	(4.A)
	Parapet Walls	870	CY	310.00	269,700	40,455	15.0	(4.A)
	Cement	28,700	Cwt	4.00	114,800	17,220	15.0	(4.A)
	Reinforcing Steel	457,000	Lbs	0.50	228,500	34,275	15.0	(4.A)
	Concrete, Upstream Face of Dam	53,200	CY	100.00	5,320,000	798,000	15.0	(4.A)
	Concrete, Downstream Face of Dam	21,800	CY	100.00	2,180,000	327,000	15.0	(4.A)
	Cement	469,800	Cwt	4.00	1,879,200	281,880	15.0	(4.A)
04.1.2.-	Concrete Dam, Overflow Section							
04.1.2.C	Concrete (RCC)							
	Concrete	2,065,000	CY	18.00	37,170,000	7,434,000	20.0	(4.E)
	Cement	3,717,000	Cwt	4.00	14,868,000	2,974,000	20.0	(4.E)
	Pozzolan	2,767,000	Cwt	2.60	7,194,200	1,439,000	20.0	(4.E)
	Contraction Joint Steel (Dam)	669,000	SF	5.00	3,345,000	670,000	20.0	(4.A)
	Waterstops (Dam)	3,000	LF	8.00	24,000	5,000	20.8	(4.A)
	Conventional Concrete							
	Spillway Surfacing	31,100	CY	140.00	4,354,000	653,000	15.0	(4.A)
	Spillway Splash Walls	2,300	CY	260.00	598,000	90,000	15.1	(4.A)
	Cement	216,300	CWT	4.00	865,200	130,000	15.0	(4.A)
	Reinforcing Steel	2,035,000	Lbs	0.50	1,017,500	153,000	15.0	(4.A)
	Concrete, Upstream Face of Dam	28,200	CY	100.00	2,820,000	423,000	15.0	(4.A)
	Concrete, Downstream Face of Dam	3,800	CY	100.00	380,000	60,000	15.8	(4.A)
	Cement	200,600	Cwt	4.00	802,400	120,000	15.0	(4.A)
	Subtotal, Construction Costs:				$ 169,224,850			
	Contingency rounded to an average of	19.4 % +/- *				$ 32,775,150		
04.1.-.-	**MAIN DAM (RCC)**			**SUBTOTAL:**		**$ 202,000,000**		

04.3.-.-	OUTLET WORKS						
	SLUICES THRU DAM						
04.3.2.-	Outlet Structure						
04.3.2.C	Concrete						
	Concrete in Place						
	Concrete	2,650 CY	320.00	848,000	130,000	15.3	(4.A)
	Cement	16,500 CWT	4.00	66,000	10,000	15.2	(4.A)
	Reinforcing Steel	263,500 Lbs	0.50	131,750	20,000	15.2	(4.A)
04.3.3.-	Conduit:						
04.3.3.C	Concrete						
	Concrete in Place						
	Pre-cast Concrete	13,650 CY	240.00	3,276,000	491,000	15.0	(4.A)
	Concrete (Backfill)	7,700 CY	80.00	616,000	92,000	14.9	(4.A)
	Cement	133,600 Cwt	4.00	534,400	80,000	15.0	(4.A)
	Reinforcing Steel	2,728,500 Lbs	0.50	1,364,250	205,000	15.0	(4.A)
	Waterstops	6,700 LF	8.00	53,600	8,000	14.9	(4.A)
	Steel liner	520,000 Lbs	2.00	1,040,000	160,000	15.4	(4.A)
04.3.4.-	Intake Structure:						
04.3.4.C	Concrete						
	Concrete in Place						
	Concrete	15,400 CY	290.00	4,466,000	2,230,000	49.9	(4.F)
	Cement	96,400 Cwt	4.00	385,600	200,000	51.9	(4.F)
	Reinforcing Steel	768,000 Lbs	0.50	384,000	200,000	52.1	(4.F)
	Trashrack						
	Concrete	200 CY	550.00	110,000	60,000	54.5	(4.F)
	Cement	1,300 Cwt	4.00	5,200	3,000	57.7	(4.F)
	Reinforcing Steel	40,000 Lbs	0.50	20,000	10,000	50.0	(4.F)
04.3.5.-	Intake Gates and Equipment:						
04.3.5.C	Concrete						
	Concrete in Place						
	Gate Chamber (Conventional conc)	13,800 CY	300.00	4,140,000	830,000	20.0	(4.A)
	Concrete, backfill (Unreinforced)	5,000 CY	80.00	400,000	80,000	20.0	(4.A)
	Cement	118,000 Cwt	4.00	472,000	94,400	20.0	(4.A)
	Reinforcing Steel	1,794,000 Lbs	0.50	897,000	200,000	22.3	(4.A)
	Access and Air Intake Structures						
	Pre-cast Concrete	5,450 CY	320.00	1,744,000	262,000	15.0	(4.A)
	Concrete (Backfill)	4,950 CY	80.00	396,000	60,000	15.2	(4.A)
	Cement	65,300 Cwt	4.00	261,200	70,000	26.8	(4.A)
	Reinforcing Steel	1,090,000 Lbs	0.50	545,000	82,000	15.0	(4.A)
	Waterstops	3,670 LF	8.00	29,360	4,000	13.6	(4.A)
04.3.5.E	Metals						
	Gate guides, embedded frame and seals						
	High Head Slide gates 5W'x9.5'H	12 Ea	60,000	720,000	110,000	15.3	(4.A)
	Slide gate frames and seals	12 Ea	10,000	120,000	20,000	16.7	(4.A)
	High Hd Bulkhead gates 7.5'Wx15'H	2 Ea	280,000	560,000	84,000	15.0	(4.A)
	Bulkhead gates Stainless steel guides, frames and seals	840,000 Lb	6.00	5,040,000	800,000	15.9	(4.A)
04.3.5.P	Conveying Systems						
	10 ton single rail crane (140'rail)	2 Each	12,500.00	25,000	4,000	16.0	(4.A)
04.3.5.Q	Mechanical						
	Gate Operating Machinery(Hydraulic)	12 Each	10,000.00	120,000	20,000	16.7	(4.A)
04.3.R.-	Associated General Items						
04.3.R.R	Electrical-Gate Chamber						
	Lighting	1 LS	50,000.00	50,000	7,500	15.0	(4.A)
	Electric Power and Lighting	1 LS	50,000.00	50,000	7,500	15.0	(4.A)
	Standby Power Equipment	1 LS	50,000.00	50,000	7,500	15.0	(4.A)
	Subtotal, Construction Costs:			$ 28,920,360			
	Contingency rounded to an average of	23.1 % +/- *			$ 6,679,640		
04.3.-.-	OUTLET WORKS		SUBTOTAL:		$ 35,600,000		

DETAILED ESTIMATE OF FIRST COST TABLE 2 C4

Main Dam

06.-.-.-	**FISH AND WILDLIFE FACILITIES**					
06.3.-.-	**WILDLIFE FACILITIES AND SANCTUARIES**					
06.3.B.-	Habitat and Feeding Facilities:					
06.3.3.B	Site Work					
	Elderberry Plantings	32,340 Each	100.00	3,234,000	320,000	
	Fencing, Barbed Wire	53,000 LF	3.00	159,000	16,000	
	Subtotal, Construction Costs:		$	3,393,000		
	Contingency rounded to an average of	9.0 % +/- *		$	307,000	(6.A)
06.3.-.-	**WILDLIFE FACILITIES AND SANCTUARIES**	**TOTAL:**		**$**	**3,700,000**	
20.-.-.-	**PERMANENT OPERATING EQUIPMENT**					
20.0.3.-	Equipment, Laboratory:					
20.0.3.L	Equipment					
	Structure Instrumentation	1 LS	2,120,000	2,120,000	530,000	
	Hydrometeorologic Instrument	1 LS	200,000	200,000	50,000	
	Subtotal, Construction Costs:		$	2,320,000		
	Contingency rounded to an average of	25.0 % +/- *		$	580,000	(20.A)
20.-.-.-	**PERMANENT OPERATING EQUIPMENT**	**TOTAL:**		**$**	**2,900,000**	
30.-.-.-	**PLANNING, ENGINEERING & DESIGN**					
30.C.-.-	LOCAL COOPERATIVE AGREEMENTS					
30.C.A.-	Draft LCA	100 DAYS	560.00	56,000		
30.C.B.-	Final LCA & Financial Plan	30 DAYS	560.00	16,800		
30.C.1.-	LCA Negotiations	10 DAYS	560.00	5,600		
30.C.Z.-	Contingencies	1 JOB LS			19,700	
30.D.-.-	ENVIRONMENTAL AND REGULATORY ACTIVITIES					
30.D.C.-	Supplemental EIS	3,087 DAYS	300.00	926,100		
30.D.2.-	401, 404, & ROD	133 DAYS	300.00	39,900		
30.D.Z.-	Contingencies	1 JOB LS			241,500	
30.E.-.-	DESIGN RELATED ENGINEERING					
30.E.1.-	Subsurface Explorations	1 JOB LS		815,000		
30.E.2.-	Sampling, Testing, & Analysis	1 JOB LS		1,850,000		
30.E.3.-	Model Development & Testing	1 JOB LS		400,000		
30.E.Z.-	Contingencies	1 JOB LS			766,300	
30.F.-.-	LETTER REPORT					
30.F.A.-	Draft Letter Report	330 DAYS	310.00	102,300		
30.F.B.-	Final Letter Report	80 DAYS	310.00	24,800		
30.F.Z.-	Contingencies	1 JOB LS			32,400	
30.G.-.-	FEATURE DESIGN MEMORANDUM (MITIGATION)					
30.G.A.-	Draft Feature Design Memorandum	710 DAYS	310.00	220,100		
30.G.B.-	Final FDM	180 DAYS	310.00	55,800		
30.G.Z.-	Contingencies	1 JOB LS			68,900	
30.G.-.-	FEATURE DESIGN MEMORANDUM (MATERIALS)					
30.G.A.-	Draft Feature Design Memorandum	1,970 DAYS	310.00	610,700		
30.G.B.-	Final FDM	490 DAYS	310.00	151,900		
30.G.Z.-	Contingencies	1 JOB LS			190,600	
30.G.-.-	SFDM (THERMAL STUDIES)					
30.G.A.-	Draft Supplemental FDM	1,600 DAYS	310.00	496,000		
30.G.B.-	Final Supplemental FDM	400 DAYS	310.00	124,000		
30.G.Z.-	Contingencies	1 JOB LS			154,600	
30.G.-.-	FEATURE DESIGN MEMORANDUM (MAIN DAM)					
30.G.A.-	Draft Feature Design Memorandum	5,520 DAYS	310.00	1,711,200		
30.G.B.-	Final FDM	1,380 DAYS	310.00	427,800		
30.G.B.-	Value Engineering Studies	290 DAYS	310.00	89,900		
30.G.Z.-	Contingencies	1 JOB LS			557,500	

30.G.-.-	OTHER (LAKE FILLING, WC MANUAL OPERATION MAINT)					
30.G.A.-	Draft Letter Report	1,950 DAYS	310.00	604,500		
30.G.B.-	Final Letter Report	490 DAYS	310.00	151,900		
30.G.Z.-	Contingencies	1 JOB LS			188,800	
30.H.-.-	PLANS AND SPECIFICATIONS - MITIGATION					
30.H.A.-	Preliminary Design	480 DAYS	310.00	148,800		
30.H.B.-	Final Design	110 DAYS	310.00	34,100		
30.H.C.-	Design Revisions	30 DAYS	310.00	9,300		
30.H.E.-	BCO Review	30 DAYS	310.00	9,300		
30.H.Z.-	Contingencies	1 JOB LS			51,300	
30.H.-.-	PLANS AND SPECIFICATIONS - RCC TEST SECTION					
30.H.A.-	Preliminary Design	640 DAYS	310.00	198,400		
30.H.B.-	Final Design	170 DAYS	310.00	52,700		
30.H.C.-	Design Revisions	40 DAYS	310.00	12,400		
30.H.E.-	BCO Review	30 DAYS	310.00	9,300		
30.H.Z.-	Contingencies	1 JOB LS			68,500	
30.H.-.-	PLANS AND SPECIFICATIONS - MAIN DAM					
30.H.A.-	Preliminary Design	3,900 DAYS	310.00	1,209,000		
30.H.B.-	Final Design	1,040 DAYS	310.00	322,400		
30.H.C.-	Design Revisions	260 DAYS	310.00	80,600		
30.H.E.-	BCO Review	160 DAYS	310.00	49,600		
30.H.Z.-	Contingencies	1 JOB LS			415,100	
30.J.-.-	ENGINEERING DURING CONSTRUCTION					
30.J.H.-	Value Engr. Change Proposals (VECP)	680 DAYS	310.00	210,800		
30.J.1.-	Review E&D Effort by Constr Contrct	340 DAYS	310.00	105,400		
30.J.2.-	Periodic Inspections	1,020 DAYS	310.00	316,200		
30.J.9.-	All Other Engineering During Constr	1,360 DAYS	310.00	421,600		
30.J.Z.-	Contingencies	1 JOB LS			263,000	
30.M.-.-	COST ENGINEERING	1,650 DAYS	310.00	511,500		
30.P.-.-	PROJECT MANAGEMENT	7,950 DAYS	400.00	3,180,000		
30.P.Z.-	Contingencies	1 JOB LS			795,400	
30.Z.-.-	MISCELLANEOUS ACTIVITIES					
30.Z.1.-	FWS Support	1 JOB LS		403,000		
30.Z.1.-	Surveys (Topographical)	1 JOB LS		460,000		
30.Z.1.-	Surveys (Cultural)	1 JOB LS		1,020,000		
30.Z.1.-	RCC Test Section	1 JOB LS		500,000		
30.Z.Z.-	Contingencies:	1 JOB LS			595,800	
	Subtotal, Construction Costs:			$ 18,144,700		
	Contingency rounded to an average of	24.6 % +/- *		$	4,455,300	(30.B)
30.-.-.-	**PLANNING, ENGINEERING & DESIGN**		**Subtotal:**	$	**22,600,000**	
31.-.-.-	**CONSTRUCTION MANAGEMENT (S & I)**					
31.B.-.-	CONTRACT ADMINISTRATION					
31.B.1.-	Pre-award Activities					
31.B.1.1	Resident Office	1,833 MH	45.00	82,485	9,000	
31.B.1.2	District Office	4,562 MH	60.00	273,720	30,000	
31.B.2.-	Award Activities	1,141 MH	60.00	68,460	8,000	
31.B.3.-	Review/Approval of Contract Payment	10,999 MH	45.00	494,955	55,000	
31.B.4.-	Contract Modifications					
31.B.4.1	Resident Office	73,329 MH	45.00	3,299,805	366,000	
31.B.4.2	District Office	5,703 MH	60.00	342,180	38,000	
31.B.5.-	Progress and Completion Reports	7,333 MH	45.00	329,985	37,000	
31.D.-.-	REVIEW OF SHOP DRAWINGS					
31.D.1	Resident Office	36,664 MH	45.00	1,649,880	183,000	
31.D.2	District Office	5,703 MH	60.00	342,180	38,000	

DETAILED ESTIMATE OF FIRST COST TABLE 2 C6

Main Dam

31.E.-.-	INSPECTION AND QUALITY ASSURANCE					
31.E.1.-	Schedule Compliance	7,333 MH	45.00	329,985	37,000	
31.E.2.-	Compliance Sampling & Testing					
31.E.2.1	Resident Office	27,498 MH	45.00	1,237,410	137,000	
31.E.2.2	Laboratory Charges	1 JOB LS		1,237,410	137,000	
31.E.9.-	Quality Assurance Personnel	93,494 MH	45.00	4,207,230	467,000	
31.F.-.-	PROJECT OFFICE OPERATION					
31.F.1.-	Resident Office	7,333 MH	45.00	329,985	37,000	
31.F.2.-	Vehicles and Equipment	1 JOB LS		3,301,781	366,000	
31.H.-.-	CONTRACTOR CLAIMS & LITIGATIONS	19,960 MH	60.00	1,197,600	133,000	
31.P.-.-	PROJECT MANAGEMENT	19,960 MH	60.00	1,197,600	133,000	
.Z.Z.-	Contingencies:					
	Subtotal, Construction Costs:		$	19,922,651		
	Contingency rounded to an average of	10.9 % +/- *			$ 2,177,349	(31.A)
31.-.-.-	**CONSTRUCTION MANAGEMENT (S & I)**		**TOTAL:**		**$ 22,100,000**	
	NON-FEDERAL					
01.-.-.-	**LANDS AND DAMAGES**					
01.B.-.-	POST-AUTHORIZATION PLANNING					
01.B.1.-	Develop Cost Estimate	25 MD	450.00	11,250		
01.B.2.-	Develop Acquisiton Schedule	23 MD	450.00	10,350		
01.B.9.-	Contingencies				2,200	
01.D.-.-	ACQUISITIONS					
01.D.1.-	Attorney's Opinion					
01.D.1.E	Review of Documents	13 MD	450.00	5,850		
01.D.1.Z	All Other	24 MD	450.00	10,800		
01.D.2.-	Mapping, Survey and Tract Ownership					
01.D.2.D	Prepare Documents	191 MD	450.00	85,950		
01.D.2.E	Review of Documents	143 MD	450.00	64,350		
01.D.2.F	Review for Compliance	72 MD	450.00	32,400		
01.D.2.Z	All Other	72 MD	450.00	32,400		
01.D.3.-	Title Evidence					
01.D.3.D	Prepare Documents	64 MD	450.00	28,800		
01.D.3.Z	All Other	32 MD	450.00	14,400		
01.D.4.-	Negotiations and Closing					
01.D.4.D	Prepare Documents	371 MD	450.00	166,950		
01.D.4.E	Review of Documents	151 MD	450.00	67,950		
01.D.4.F	Review for Compliance	76 MD	450.00	34,200		
01.D.4.Z	All Other	89 MD	450.00	40,050		
01.D.5.-	Condemnation (Pre-DT Filing)					
01.D.5.D	Prepare Documents	32 MD	450.00	14,400		
01.D.5.E	Review of Documents	13 MD	450.00	5,850		
01.D.5.F	Review for Compliance	6 MD	450.00	2,700		
01.D.5.Z	All Other	6 MD	450.00	2,700		
01.D.9.-	Contingencies				61,000	
01.E.-.-	CONDEMNATION (POST-DT FILING)					
01.E.0.D	Prepare Documents	26 MD	450.00	11,700		
01.E.0.E	Review of Documents	7 MD	450.00	3,150		
01.E.0.F	Review for Compliance	6 MD	450.00	2,700		
01.E.0.Z	All Other	115 MD	450.00	51,750		
01.E.9.-	Contingencies				6,900	

Main Dam

01.F.-.-	APPRAISALS				
01.F.1.-	Staff Appraisals				
01.F.1.H	Prepare Documents	239 MD	450.00	107,550	
01.F.1.J	Review of Documents	96 MD	450.00	43,200	
01.F.1.F	Review for Compliance	96 MD	450.00	43,200	
01.F.1.Z	All Other	48 MD	450.00	21,600	
01.F.2.-	Contract Appraisals				
01.F.2.H	Prepare Documents	29 MD	450.00	13,050	
01.F.2.J	Review of Documents	29 MD	450.00	13,050	
01.F.9.-	Contingencies				24,200
01.K.-.-	TEMPORARY PERMITS				
01.K.0.D	Prepare Documents	26 MD	450.00	11,700	
01.K.0.Z	All Other	13 MD	450.00	5,850	
01.K.9.-	Contingencies				4,400
01.M.-.-	REAL ESTATE RECEIPTS/PAYMENTS				
01.M.3.-	Land Payments	1 LS		9,288,000	
01.M.5.-	Damage Payments	1 LS		929,000	
01.M.9.-	Contingencies				2,322,000

Subtotal, Construction Costs: $ 11,176,850

Contingencies @ average of 21.7 % +/- * $ 2,423,150 (1.B)

01.-.-.- LANDS AND DAMAGES TOTAL: $ 13,600,000

SUMMARY OF ANNUAL COST **TABLE 3**

ITEM	COST $
Price Level as of 1 Oct-91	
	8.750%
A. INVESTMENT COST	
1. FEDERAL	
TOTAL	422,900,000
2. NON-FEDERAL	
TOTAL	267,660,000
TOTAL PROJECT INVESTMENT	**690,560,000**
B. ANNUAL COSTS	
1. FEDERAL	
TOTAL	37,000,000
2. NON-FEDERAL	
TOTAL	24,700,000
TOTAL PROJECT ANNUAL COST	**61,700,000**

DETAILED ESTIMATE OF ANNUAL COST TABLE 4

ITEM		COST $
Price Level as of 1 Oct-91		
		8.750%
A. INVESTMENT COST		
1. FEDERAL		
a. First Cost		380,000,000
b. Interest During Construction		42,900,000
TOTAL		**422,900,000**
2. NON-FEDERAL		
a. First Cost		240,460,000
b. Interest During Construction		27,200,000
TOTAL		**267,660,000**
TOTAL PROJECT INVESTMENT		**690,560,000**
B. ANNUAL COSTS		
1. FEDERAL		
a. Interest Rate	8.750%	37,000,000
TOTAL		**37,000,000**
2. NON-FEDERAL		
a. Interest Rate	8.750%	23,400,000
d. Maintenance And Operation:		1,200,000
2 Relocations	400,000	
3 Reservoirs	200,000	
4 Dams	200,000	
13 Pumping Plants	200,000	
14 Recreation Facil.	100,000	
20 P. O. Equip.	100,000	
e. Replacement Costs:		100,000
20 P. O. Equip.	100,000	
TOTAL		**24,700,000**
TOTAL PROJECT ANNUAL COST		**61,700,000**